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THE INFLUENCE OF PORE WATER SALT CONTENT ON THE
SHEAR STRENGTH OF A REMOULDED HOMIIONIC CLAY

by

J. G. LOCKER

A THESIS

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled "The Influence of Pore Water Salt Content on the Shear Strength of a Remoulded Homionic Clay" submitted by J. G. Locker in partial fulfilment of the requirements for the Degree of Master of Science.

ABSTRACT

Based on previous research at the University of Alberta, an investigation of the variation in shear strength of sodium and calcium homionic modifications of a remoulded clay with varying concentrations of salts in the pore water was performed. The shear strengths were determined by the means of consolidated undrained triaxial compression tests with pore pressure measurements. Pore water salt contents were 0.2, 2.3, 4.7, 7.6, and 13.1 me/100 gms. a.d.s. for the sodium modification, and 13.0, 18.9, and 68.3 me/100 gms. a.d.s. for the calcium modification. A detailed description and discussion of all operations is included. The results are presented in tabular and graphical form to show the changes in geotechnical properties with salt content.

The angles of shearing resistance for the sodium modification increase with the increase in salt content to about 8 me/100 gms. a.d.s. and then remain relatively constant; however, with the calcium modification there is no significant change within the range of salt contents studied. Time to theoretical 100% consolidation decreases markedly with an increase in salt content for the sodium modification, but shows a slight increase for the calcium modification with an increase in salt content. Liquid limit decreases with an increase in salt content for both modifications, but the plastic limit remains relatively constant.

The concentration of salts in the pore water influences the thickness of the adsorbed water hull and this variation has been used to explain the variations in soil properties. Consideration is given to an explanation of the shear strength variations by Lamb's equation. Further research should involve other adsorbed complexes and systems.

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GLOSSARY OF TERMS

- A -Electrical attraction between particles
- \bar{A} -Pore pressure parameter; used herein to designate the ratio of pore pressure to the deviator stress
- Angle of Shearing Resistance -The angle which defines the slope of the Mohr rupture envelope.
- B -Emperical parameter; used herein to represent the change in pore pressure due to a change in confining pressure, i.e., the pore pressure reaction test.
- Bonding Energies - Energies that are capable of bonding charged elements to a particle
- c -Cohesion on the total stress Mohr envelope
- c' -Cohesion on the effective stress Mohr envelope
- CEC -Cation Exchange Capacity: The total quantity of cations in me/100 gms. a.d.s., as determined by the flame photometer and includes both those cations adsorbed and those present in the pore water.
- Cohesion -Used herein as the intercept of the Mohr rupture envelope with the vertical axis (shear strength axis)
- Double Layer -The double layer is composed of an internal monolayer and an outer diffuse layer (Sec. 6:2)
- e -Void Ratio
- G_s -Specific gravity of the soil mass, includes all salts present.
- Homiõnic -When all the exchange positions of a clay mineral are satisfied by one type of cation, e.g. Na⁺
- Macroscopic -Visible to the naked eye
- me/100 gms. a.d.s. -Milliequivalents per 100 grams of air dried soil. A milliequivalent is the amount of a reagent required to combine a react with 1/1000th of an atomic weight of hydrogen
- N.G.I. -Norwegian Geotechnical Institute

Normal	-A normal solution contains one gram-equivalent per litre
P_L	-Plastic limit of the soil mass, includes all salts present.
Plasticity Chart	-A plot of plasticity index versus liquid limit; as presented by A. Casagrande
Polarizability	-The ability to deform charges from spherical symmetry to give rise to polar action.
Pore Water	-All the free water in the clay-water system; i.e., excludes the adsorbed water.
P_p	-Pore pressure in the pore water of the soil.
R	-Electrical repulsion between particles
S.C.	-Salt Content: The concentration of salts in the pore water expressed in me/100 gms. a.d.s., and has been taken as being equal to the numerical difference between the CEC and the TEC.
Sensitivity	-The degree of sensitivity of a clay is expressed by the ratio between the unconfined compressive strength of an undisturbed specimen and the strength of the same specimen at the same water content but in a remoulded state.
Shear Strength	-The strength of a soil as defined by the concept of Mohr's envelope and composed of an angle of shearing resistance and a cohesive value.
Specific Surface	-Area of a material per unit mass, e.g. cm^2/gm .
TEC	-Total Exchange Capacity: The quantity of cations adsorbed by the clay mineral in me/100 gms. a.d.s. as determined by the titration procedure.
Surface Charge Density	-The charge present on the surface of a clay particle per unit of area
t_{100}	-Time required to reach theoretical 100% consolidation as determined by the graphical construction from the logarithm of time fitting method.
t_{90}	-Time required to reach theoretical 90% consolidation as determined by the graphical construction from the square root of time fitting method.
W_l	-liquid limit of the soil mass, includes all salts present
Zeta Potential	-The difference between the potential of the immoveable layer attached to the surface of the solid phase and the potential of the free water
\emptyset	-total angle of shearing resistance (degrees)

ϕ'	-effective angle of shearing resistance (degrees)
σ_1	-total major principal stress(Kg/cm ²)
σ_1'	-effective major principal stress (Kg/cm ²)
σ_3	-total minor principal stress(Kg/cm ²)
σ_3'	-effective minor principal stress (Kg/cm ²)
$\sigma_1 - \sigma_3$	-deviator stress (Kg/cm ²)

CHAPTER I

INTRODUCTION

1:1 General

The general behavior of soils is of interest to the soil scientist, the geologist, and the soils engineer. It has been more apparent than ever in recent years, that each of these disciplines should cooperate in order to obtain a more comprehensive understanding of the nature of soil. Soils engineers today are attempting to explain the macroscopic^{*1} behavior of cohesive soils by studying microscopic phenomena (Lambe, 1960; Rosenqvist, 1959; Warkentin, 1962)².

Fine grained soils lend themselves to study of physico-chemical phenomena on a microscopic scale due to the electrical unbalance and the high specific surface^{*} of the crystalline clay minerals. The electrical unbalance allows the clay mineral to adsorb certain cations and anions and retain these in an exchangeable state, that is, the adsorbed ion on a clay particle may be exchanged by treatment with other ions in an aqueous solution.

There are several reasons why the engineer should be concerned with cation exchange. The properties of cohesive soil have been shown to vary with the nature of the exchangeable ion, (Hamilton, 1961; Thomson, 1963; Grim, 1942). Knowing the possible changes in the geotechnical properties

¹ * indicates expressions explained in glossary

² References listed alphabetically in "List of References".

when chemically modified and knowing the properties desired in the soil mass, it appears feasible that the soils engineer should be able to tailor the soil to some extent to fulfill his needs. Winterkorn (1953,p.37) relates incidents of failures which occurred due to changes in soil properties as a result of ion exchange during and following construction. The nature of the exchangeable ion may affect the reactivity of the soil with other substances, e.g., it influences the reaction with Portland cement in cement stabilized soils.

1:2 Research at the University of Alberta

Research on physico-chemical aspects of fine grained soils began at the University of Alberta in 1960. The preliminary research of P. A. Thomson (1960), who investigated the effects of exchangeable cations on some of the properties of clay shales, illustrated the requirement for homionic soils^{*}. The presence of extraneous cations tended to mask the effects of the cation under study. Following P. A. Thomson's recommendations, Hamilton (1961) prepared homionic modifications of a clay soil, and studied the effect of exchangeable cations on liquid limits, plastic limits, and consolidation characteristics. In general he found, that within the limits of the testing equipment, an homionic soil could be produced. He postulated that the type of structure exhibited by a modification depended upon the exchangeable cation present. The liquid and plastic limits and their relative positions on the plasticity chart^{*} were shown to be influenced by the exchangeable cation.

Investigations were continued by S. Thomson whose thesis involved an investigation of the influence of physico-chemical phenomena on the

shear strength^{*} of a normally consolidated remoulded Alberta clay. The soil was modified by having a high proportion of the exchange complex occupied by a single cation species. The adsorbed cations were calcium magnesium, potassium and sodium. Shear strength was determined on remoulded specimens by the means of the consolidated quick test with pore pressure measurements. Grain size analyses and specific gravity determinations were performed. Qualitative determinations of the total exchange capacity and the concentration of salts in the pore water were made.

The shear strength characteristics of a clay were found to be affected by the adsorbed cation when the clay is in an homionic state, particularly for the sodium modification. The adsorbed cation influenced the specific gravity, the grain size distribution and the time for theoretical 100% consolidation. The main factor considered to account for the variations was the change in thickness of the adsorbed water hull around the clay particle due to the nature of the adsorbed cation.

It was found also that a decrease in the concentration of salts in the pore water caused: (1) an increase in the time for theoretical 100% consolidation, (2) a decrease in the shear strength. No attempts were made, however, to control the concentration of salts in the pore water.

On the basis of these findings, the topic for this thesis was established as outlined in the following paragraphs.

1:3 Problem

The subject of this programme is: to investigate the variation in shear strength of two homionic clays subjected to varying concentrations of salts in the pore water^{*}. Sodium-clays with varying concentrations of

sodium acetate in the pore water and calcium-clays with varying concentrations of calcium acetate in the pore water were selected. The sodium-clay was used as it is representative of a clay type having a maximum reaction. The calcium-clay was chosen, as calcium is the predominate cation in local clays.

1:4 Scope of the Investigation

The same soil as used in previous programmes (Hamilton, 1961; Thomson, 1963) was modified to produce sodium and calcium homionic soils. Attempts were made to eliminate extraneous adsorbed cations and control the concentration of salts in the pore water. Remoulded samples of each clay modification were made with the pore water varying with respect to its concentration of salt.

Shear strengths as defined by Mohr envelopes were determined by the means of consolidated quick tests with pore pressure measurements. Liquid limits, plastic limits, and specific gravity determinations were also made. No review of clay mineralogy or soil physics was presented in this thesis; the reader is referred to either S. Thomson (1963, Appendices, B, C, D) or to other available literature.

1:5 Soil Physics and Engineering

The use of homionic soils in research may be open to question since they are not generally found in nature. However, it has been stated (Winterkorn, 1952, p.37) that it is not necessary to work with homionic soils in the field, e.g., sodium-soil characteristics can be achieved with about 15% sodium saturation of the exchange complex.

In research, when one is investigating a particular question, all other variables should be eliminated or minimized. As a step towards the study of natural soils, one must first determine the characteristics of a pure state and then go to more complex systems. Therefore, despite the fact that it is difficult to produce 100% homionic soils, it seems scientifically correct to attempt to do so in order to eliminate extraneous factors.

When studying the effects of physico-chemical phenomena upon the engineering properties of soil, the student discovers himself involved with different levels of structure, i.e., macroscopic, microscopic, atomic, etc. This is due to the fact that research is being conducted and interpreted by different levels of the engineering and scientific mind. This particular investigation has been conducted in the light of previous research at the University of Alberta, i.e., accepted engineering test procedures have been employed and accompanying discussions are presented from an engineering point of view aided by the knowledge of the soil scientist where required. In general, the results and postulations of this work have been presented in a qualitative manner. Winterkorn (1952,p.29) states with respect to soil stabilization: "Qualitative thinking is of paramount importance and premature exclusive emphasis on the quantitative may well stymie the growth of this important discipline."

CHAPTER II

LITERATURE REVIEW

Recent research relating physico-chemical phenomena to soil mechanics presents some difficulties in correlation due to variations in soil type, soil structure, and testing procedure.

Studies have been made on clay-water suspensions, homionic soils, and naturally occurring soils. A natural soil used to investigate a particular problem, e.g., effect of electrolyte concentration, introduces serious unknowns with respect to quantities and types of ions either adsorbed or present in the free water. Also it does not seem feasible to directly compare the results of a research programme concerned with clay-water suspensions to that of the plastic clay-water mass normally associated with soil mechanics. (Lambe, 1958, p.1654-10).

The evaluation of shear strength is an exceedingly complex problem due to the number of variables involved. A correlation of shear strength as determined by such techniques as direct shear, vane shear, drop cone, unconfined compression, and triaxial testing is difficult when encountered in regular research problems, thus when coupled with physico-chemical effects additional difficulties arise.

Therefore, it becomes difficult to directly compare most literature to a research programme of this nature where homionic soils and triaxial testing was involved.

Behavior of the microscopic state has been viewed in the light of interparticle forces by Lambe (1960) and Warkentin (1962), polarizability* of the adsorbed cations by Rosenqvist (1955), and structure by Lambe (1958, No. 1654). There are numerous postulates but there appears to be a common thread running throughout the ideas, and it is hoped that research will tie them together to form a comprehensive picture.

The topic of the author's research has been mentioned in the literature in numerous instances (Lambe, 1958,p.1654-30; Rosenqvist, 1955,p.70), however, no programmes of the particular nature undertaken herein were found.

Lambe (1958,p.1654-30) states:

"Any change in the soil-water system which expands the double layer* tends to decrease soil strength (at a given void ratio), since interference increases inter-particle repulsion. This concept leads to the prediction that any of the following changes would generally reduce the shear strength of a clay:

1. Reduction of electrolyte concentration
2. Cation exchange from high to low valence (e.g., Ca^{++} to Na^+)
3. Exchange from a cation of small hydrated radius to a cation of large hydrated radius (e.g., Na^+ to Li^+)
4. Adsorption of anions, (e.g. phosphate)
5. Increase of dielectric constant of pore fluid
6. Increase of pH of pore fluid
7. Decrease of temperature
8. Increase in water content

The influence of some of these eight changes has been investigated. The Norwegians (Bjerrum 1954; Rosenqvist, 1955; and Bjerrum and Rosenqvist, 1957) have, for example, conclusively shown that a reduction of electrolyte by leaching can reduce both the undisturbed and remoulded shear strengths."

Rosenqvist (1955) investigated the effect of varying concentrations of electrolyte upon clay-water suspensions using Norwegian marine and quick clays. Shear strength, sensitivity*, and Atterberg limits were

determined. No mention was made of the shear strengths being determined in terms of effective stresses. As a homionic soil was not used, it is likely that the ion equilibrium was disturbed, thereby preventing controlled conditions. It was concluded that the important effect, the increase in salt content had on the engineering properties of the soil, was due to its effect on the zeta potential* of the clay particle.

Rosenqvist and Bjerrum (1957, p.9) working with artificially sedimented clays reported reductions in shear strength of 40-50% with the leaching out of the salts in the pore water. The shear strength was determined in terms of undrained shear strength and drained shear strength. It was shown that it is possible to produce in the laboratory, artificially sedimented clays which are very similar to natural clays. Using the same techniques, Feyling-Hanssen (1957, p.31) reported: "The reduction in geotechnical properties does not take place in linear proportion to the reduction in salt concentration".

Lambe (1958, No.1654) has studied compacted clays and has attempted to formulate a trend of thought which would stimulate further research. He makes reference to the fact that a decrease in electrolyte concentration results in a decrease in shear strength and that this fact is related to the thickness of the double layer.

Lambe's "Mechanistic Picture of Shear Strength in Clay" (1960) relates the shear strength of clay to the forces present between clay particles. The wet clay particle was presented as a mineral particle surrounded by a double layer consisting of mobile ions in water. Temperature, concentration, and type of ions in the double layer and dielectric properties of the double layer fluid were considered to be relevant to the size and nature of the double layer.

The normal force between clay particles was related to the shear resistance between them, and may be represented by an equation which considers the forces between mineral, water and air phase contacts and the electrical attractive and repulsive forces. Lambe considers mineral-mineral and air-mineral contacts to be negligible if the material is saturated. All water, even that adsorbed, is considered to be able to transmit only hydrostatic pressures. The repulsive forces related to Coulombic pressure and attractive forces due to van der Waals forces were considered to be active.

The equation is:

$$\sigma_n = \sigma_T - (u + R - A)$$

where:

σ_n denotes effective normal Stress

σ_T denotes total stress

u denotes pore pressure in water phase

R denotes electrical repulsion between particles

A denotes electrical attraction between particles

This equation aids in explaining shear strength but cannot be applied practically as not all factors can be evaluated at present. Lambe suggests that future research to explain shear mechanisms should involve triaxial testing with pore pressure measurements.

Warkentin (1962, p.1) states:

"The action of forces between clay particles can explain the behavior of soils in certain engineering applications. These forces are associated with the high surface area and electrical charge of clays, and their action depends upon the composition and arrangement of the particles and upon the pore fluid."

The shear strength of sodium and calcium montmorillonite with two different salt concentrations in the pore water was investigated by the

means of the direct shear test. In both cases the shear strength of the soil with the highest concentration of salts in the pore water had the lowest shear strength when compared at equal void ratios (Warkentin, 1962).

Some researchers have shown a preference for working with a soil comprised of a single clay mineral. Quigley (1962) studied the engineering properties of illite with respect to the pore water and related them by a fabric study. The fabric study revealed that shearing produces marked particle orientation, and that the sodium clay developed a more parallel structure during consolidation than the potassium clays.

Olson and Mitranovas (1960) examined the effects of variations in the pore water electrolyte concentration on the engineering properties of homionic calcium and magnesium illite. The Atterberg limits were affected slightly by the electrolyte concentration in the range of concentrations of 0.1 normal* (N) to 0.01 N. The shear strength as determined in terms of effective stress from the triaxial tests showed no variation with a change in salt concentration. The authors felt that the most important effect of the electrolyte concentration was its influence on the geometric arrangement of particles for samples which were sedimented from a dilute suspension. A very interesting feature of this testing programme was the preparation of the triaxial specimens by a special sedimentation procedure.

The literature generally agrees that a reduction in shear strength will be caused by a reduction in the electrolyte concentration in the pore water. However, the author was unable to locate any controlled research using a homionic mixed clay mineral and quick consolidated tests with pore pressure measurements.

CHAPTER III

SAMPLE PREPARATION AND TESTING PROGRAMME

3:1 Soil

The fine grained soil employed in this programme was the same as that used by Hamilton (1961), Thomson (1963), and Semchuk (1962). The soil originally came from a highway cut which was opened in the latter part of the summer of 1959 on the southern outskirts of Edmonton, Alberta. In early April 1960, the surface material was cleared away, and about 500 pounds of soil were dug out from a point about 15 to 20 feet below the original ground surface, i.e., well below the zone of weathering. The soil was stored in the laboratory where it was allowed to air dry. Geologically, the material is a glacial lake sediment and is classified as a highly plastic clay. It has a total exchange capacity of approximately 33 me/100 gms. a.d.s.*

Classification tests in accordance with the procedures for testing soils as outlined by the American Society for Testing Materials and designated as: Specific Gravity D854-52, Liquid Limit D423-54T, Plastic Limit D424-54T and Grain Size Analysis D422-54T were performed. The clay mineralogical composition of the clay fraction was determined by the Alberta Research Council. The results of these tests are presented in Table I.

TABLE NO. I¹

SUMMARY OF CLASSIFICATION TESTS ON ORIGINAL CLAY

Test	Result
Specific Gravity	2.80
Atterberg Limits	
Liquid Limit	74.3
Plastic Limit	30.8
Plasticity Index	43.5
Grain Size Distribution ²	
% Sand Sizes	4
% Silt Sizes	34
% Clay Sizes	62
Mineralogical Composition ³	
Montmorillonite	30 - 40%
Illite	30 - 40%
Chlorite	20 - 30%

¹ From Semchuk (1962, p.15) with permission

² M.I.T. Grain Size Scale

³ Rough approximation; furnished by Research Council of Alberta.
On clay size fraction only

3:2 Soil Modification

The programme required a modification of the original clay to yield two homionic soils; a sodium-clay and a calcium-clay. The methods used to produce the homionic modifications were based on previous research at the University of Alberta, (Hamilton, 1961; Thomson, 1963).

The original clay was broken down to pass a No. 40 sieve by the means of a mechanical mortar and pestle. The material was divided into 1500 gm. batches and placed into two gallon polyethylene buckets. Large containers were used as it was believed that much time would be saved and more uniform modifications would result by treating larger portions of soil. The following treatments were performed on the portion in each bucket.

Carbonates and sulphates were removed from the natural soil by slowly adding four liters of 0.75 N hydrochloric acid (HCl) and stirring. It was believed that HCl of this concentration would not damage the clay fraction to any significant degree. After standing for one day, the soil had settled out and the supernatant solution was siphoned off. This entire operation was repeated three more times for a total of four washings with the acidic solution. The soil mixture was then washed four times in a similar manner with distilled water. At this stage of the preparation it was assumed that this treatment was sufficient to remove the carbonates and sulphates.

In order to bring about the exchange required to produce a homionic soil, a 1.0N solution of the acetate of the desired cation was mixed with the soil. The ratio of acetate to soil was 10:1. Each of the four washings was well stirred and allowed to stand for one or two days until

the soil was settled out, and the supernatant solution could be siphoned off.

Each bucket of soil was divided into four to six one-quart sealers. The excess salts were removed by washing the treated soil with ethyl alcohol. Each sealer of soil was split into eight 250 millilitre centrifuge bottles, and mixed with ethyl alcohol by the means of a milkshake mixer. The soil was brought out of solution by centrifuging at 1500 rpm for about 10 minutes or until the soil settled out. The solution was then decanted. Sufficient ethyl alcohol was used each time so that four or five washings would result in about 25 to 35 grams of soil being washed with approximately 200 to 300 milliliters of ethyl alcohol. It was assumed at this time that this treatment was sufficient to remove the excess salts. The soil was then placed into an evaporating dish and air dried. The material was broken down to pass a number forty sieve and all batches were mixed together to provide a uniform soil.

3:3 Natural Clay

Triaxial samples were made from the original clay after it had been subjected to the normal washing procedure using the HCl solution. This treatment removed the carbonates and sulphates but otherwise did not change the characteristics of the original soil. Consolidated quick triaxial tests with pore pressure measurements were performed on three samples in order to form a Mohr envelope.

3:4 Moulding the Triaxial Test Specimen

Sufficient soil for at least three triaxial specimens was mixed in one batch with an adequate amount of a solution of a desired normality to form a thick slurry with a moisture content slightly in excess of the

liquid limit. The solutions were of varying normalities of the acetate of the desired cations as noted below.

Sodium-clay with 0.0N, 0.1N, 0.2N, 0.3N, 0.5N salts in the pore water

Calcium-clay with 0.0N, 0.3N, 0.8N salts in the pore water

The slurries were well mixed; the container covered with aluminum foil and placed in the moist room for one day. The slurry was remixed before placing it into the triaxial specimen moulds. The moulding apparatus consists of two concentric lucite cylinders. The inner cylinder into which the soil was placed, has four filter strips (Whatman No.54) placed longitudinally on the inner periphery with their lower ends in contact with the filter paper and the bronze porous disc. The bronze porous disc was inserted into the end of the cylinder to allow drainage. The soil was placed into the cylinder with the aid of a spatula and the cylinder was tapped on the counter to remove any entrapped air. A perforated loading cap with a filter paper and bronze porous disc was placed on the sample. The loading cap is perforated to allow drainage. This entire assembly was placed inside the larger outer cylinder. The same solution as was used to prepare the slurry was used to soak all filter papers and porous discs, and to fill the annular space between the cylinders.

By the means of a lever arm and weights, a static axial load was applied to the sample. An extensometer was used to obtain time curves which could be plotted in the same manner as consolidation tests. An initial load of approximately 0.3 Kg/cm^2 was applied in order to allow the specimen to consolidate sufficiently so that the final load of 1 Kg/cm^2 would not allow squeezing of the soil past the loading cap. Under the 1 Kg/cm^2 load the specimen was allowed to consolidate until the time curve indicated that

theoretical 100% consolidation had taken place.

The apparatus was dismantled and the specimens were extruded by applying a pressure to the loading cap in the direction of load application. The samples were carefully wrapped in aluminum foil, thoroughly waxed, and stored in an air tight container in the moist room until required.

3:5 Mounting the Triaxial Test Specimen

The procedure followed in mounting the specimen is similar to that used by the Norwegian Geotechnical Institute (Andresen et al, 1957). A detailed description of the procedure followed in this testing programme can be found in Appendix B. The specimen was trimmed to 35.7 mm in diameter by 80 mm long. Five wool wicks were inserted into the sample and the dimensions and weight of the sample were noted. The sample was mounted in the triaxial cell and protected by two rubber membranes and two layers of silicone grease.

A confining pressure was applied and the sample was allowed to consolidate by expelling water into a 25 millilitre stopcock burette. A consolidation time curve was plotted from burette readings at appropriate time intervals. After consolidation the burette was removed and the triaxial cell was mounted on the loading press.

3:6 The Triaxial Test

The procedure used to perform a triaxial test is outlined in detail in Norwegian Geotechnical Institute publications (Andresen et al, 1957; NGI. No.45, 1957), therefore, the following description is brief.

Taking care to properly flush all air out of the system, the pore pressure measuring device is hooked into the system. A back pressure of

2 Kg/cm^2 was introduced into the system and allowed to act overnight. The back pressuring system is discussed further in the discussion of the testing procedure (Sec. 4:6). Immediately preceding the strength test a pore pressure reaction test was conducted. This test consists of increasing the cell pressure 1 Kg/cm^2 simultaneously with starting a stop-clock and observing the pore pressure increase with time. Normally after five minutes the additional cell pressure was released and the decrease in pore pressure measured with time.

The position of the proving ring and strain gauges was checked and the loading commenced. Readings of stress, strain, time and pore pressure were taken at increments of strain of 0.10% up to 2%, 0.20% up to 3%; 0.25% up to 4% and 0.50% up to failure. As the test progressed values of deviator stress, major and minor principal effective stress, effective principal stress ratio and pore pressure parameter, \bar{A}^* , were calculated. These values and pore pressure were plotted versus per cent strain.

The axial load was applied for 2 to 4% strain past failure. Failure was considered to have occurred at maximum principal stress ratio.

At the completion of the test the entire assembly was dismantled taking care to maintain equilibrium conditions within the specimen. The following factors were noted: weight, volume (by mercury immersion), mode of failure, and dimensions. A complete example of the data for one triaxial test can be found in Appendix C. The triaxial specimen was cut into three portions: one (center) for moisture content, one for flame photometer, and one for specific gravity determination.

CHAPTER IV

DISCUSSION OF SAMPLE PREPARATION AND TESTING PROCEDURE

4:1 General

The problem of this thesis involved one major variable, i.e., a study of the effects of a variable salt content upon two modified soils. Therefore, the utmost of care was taken to eliminate variables in sample preparation and testing procedure so that the results could be compared effectively.

4:2 Soil Preparation

The use of a homionic soil practically eliminates any variation with respect to the amount and kind of cation adsorbed or present in the free water. In the preparation of the homionic soils the portions of soil being treated were kept relatively large in order to provide uniform treatment. The sodium and calcium modifications were prepared following previous research (Hamilton, 1961; Thomson, 1963) which was based on the Langmuir Adsorption Theory. The theory postulates the formation of chemical bonds for liquid-solid interfaces and the termination of adsorption when the bonds are satisfied. This phenomena has been termed "chemisorption". The relationship resulting from Hamilton's work (1961) confirms the application of the theory to cation exchange and may be used in reverse to determine the normality of a solution required to produce a homionic soil. The Law of Mass Action has been considered to hold for the purposes of this programme.

From a consideration of the following equation:



it is apparent that the production of sodium-clay is aided by the removal of calcium ions from the right hand side of the equation. In the preparation of homionic soil the supernatant solution was siphoned off and fresh salt solution added. In this manner the Law of Mass Action was applied. The results of the flame photometer tests indicated that the use of a 1.0N solution of the desired acetate and a ratio of solution to soil of 10:1 was satisfactory. The modified soil were on the average 97.3% saturated with a given cation. These have been termed homionic for this work.

Ethyl alcohol was used to remove the excess salts from the pore water. Flame photometer and titration tests on the triaxial test samples which were formed with distilled water and sodium-clay showed zero salts in the pore water. This indicated that these washings were successful. In the case of the calcium-soil, however, the washings proved to be insufficient as the same procedure revealed 13.0 me/100 gms. a.d.s. in the pore water.

The acetate radical was used as it is large and may not be readily adsorbed but its major advantage was its solubility in ethyl alcohol which facilitates its removal in the washing procedure.

Flame photometer and titration results revealed each concentration of salts in the pore water of the sodium clay to be lower than was expected from the original normality used. This was due to the fact that the water of hydration of the acetate salt was not considered when the desired weight of chemical was determined for each normality. Table II relates the normality calculated to the actual me/100 gms. a.d.s.

TABLE II

RELATIONSHIP OF ORIGINAL NORMALITIES TO MILLI-EQUIVALENTS

PER 100 GRAMS OF AIR DRIED SOIL

Series	Original Normalities	me/100 gms a.d.s. ¹
Sodium	0.0	0
	0.1	2.3
	0.2	4.7
	0.3	7.6
	0.5	13.1
Calcium	0.0	13.0
	0.3	18.9
	0.8	68.3

¹ As determined from the difference of the flame photometer and titration results.

The significance of this oversight can be illustrated by the following example calculation:

EXAMPLE:

Sodium Acetate - $\text{CH}_3\text{COONa} \cdot 3\text{H}_2\text{O}$

Weight of CH_3COONa equals 82.041 grams

Weight of $3\text{H}_2\text{O}$ equals 54.048 grams

Total weight 136,089 grams

. . the normality of a proposed 0.1N solution was actually

$$\frac{8.2041}{13.6089} \times 0.1 = 0.0628\text{N}$$

Thus it can be seen that there is a high reduction in normality due to this error.

4:3 Triaxial Sample Moulding

The use of modified soils necessitated the making of remoulded soil samples. The soil required for the preparation of three samples for the development of one Mohr envelope was mixed with the desired salt concentration in one batch in order to provide uniformity. The triaxial samples were moulded from a moisture content slightly higher than the liquid limit under a vertical pressure of approximately 1 Kg/cm^2 ; thus, the samples were considered to be normally consolidated. It was hoped that they would be identical.

In previous work by Olson and Mitranovas (1960) a special sedimentation procedure was used to form the samples. This procedure may provide more identical samples with respect to structure, however, it would not be practical to employ this method when dealing with sodium-clays. The length

of time involved to produce a sample would be tremendous due to the highly dispersed nature of a sodium-clay suspension.

It may be noted from Table IV¹ that the initial void ratios compare remarkably well for each series of samples, therefore the sample moulding technique appears quite adequate to prepare identical samples.

After the formation of the sample, it was wrapped in aluminum foil for storage. It was later thought that some exchange may have taken place with the aluminum ions of the foil. However, it is reasonable to assume that if any exchange took place it would likely be on the outer surface of the specimen, and this material is removed when trimming the specimen for the triaxial test. In order to obviate any future question in this regard, it is suggested that a layer of wax paper be placed around the sample prior to the aluminum foil.

A conductivity test performed on the distilled water used in this research by the Oil and Gas Conservation Laboratory revealed the conductivity to be 0.58×10^5 ohms at 25°C, therefore good distilled water was used.

4:4 Drainage During Triaxial Consolidation

In order to aid drainage during consolidation slotted filter paper and wool wicks were used. Both of these aids were placed in contact with the porous ceramic disc as previously described. The slotted filter paper is suggested by Norwegian workers and its merit is readily understood.

Due to the pattern in which the wool wicks are inserted, the longest drainage path is about 0.5 cm. The wool wicks aid in drainage for

¹. Table IV is a summary data sheet

consolidation and also assist in pore pressure equilization. The disturbance due to the insertion of the wicks is an undesirable feature but only a very small volume, (approximately 0.4% of original volume), is affected and the subsequent consolidation action overcomes most of the detrimental effects. As each sample was treated identically, the effects should be indiscernible when comparing final results.

4:5 The Triaxial Test

All factors concerning strain are based on the original length of the sample as correction curves established by the Norwegian Geotechnical Institute are based on original length. This correction curve involves axial strain and volume change during consolidation. The correction values obtained aid in the calculations of corrected cross-sectional area and the deviator stress. The details of this correction curve are presented in Norwegian Geotechnical Institute Publications (Andresen et al, 1957, NGI No.45, 1957).

The rests were performed at room temperature which is relatively constant. It has been shown by recent research at the University of Alberta that temperature has little effect on the shear strength of the original soil used in this work (Semchuk 1962, p.119).

The proving ring used to measure the deviator stress had a load capacity such that failure occurred within the central range which is considered to be the most accurate portion.

Due to the fact that some of the samples were in the cells for six to eight weeks, it was necessary to take precautions against air and water migration from the cell chamber into the sample. Two membranes with a layer of silicone grease between them and another layer on the outside

of the membranes appeared to fulfill the requirements as is illustrated by the facts that the samples maintained approximately their degree of saturation and the consolidation time curves were regular.

All filter paper, porous plates, wool wicks, etc. that were placed in contact with a sample were thoroughly soaked in the same salt solution as was used to prepare the sample. However, the effect of the distilled water in the pore pressure line was not considered until the latter stages of the testing programme. Hardy and Dyregrov (1962, p.9) state: "Pore pressure measurements may well be suspect for accuracy if distilled water is permitted to gain access to the sample close to the point of the pore pressure measuring tip". It is believed that this action may cause incorrect pore pressure measurements due to the influence of osmotic pressure between the distilled water and the electrolyte. Despite this fact, the remainder of the tests were performed as usual as it was felt to be inappropriate to introduce a possible variable. The possibility exists that a great deal of the salts remained in the porous stone after the soaking so that the solution in contact with the sample would have a salt content nearly identical to that of the sample. No method is known to the author as to how one could evaluate the effects without performing many additional triaxial tests. The factor was therefore neglected in the evaluation of the data.

After all the required data was obtained from the failed specimen, it was sliced into three parts. The central portion was used to obtain the moisture content of the specimen and the other two portions were used to determine the specific gravity and the salt content. A great deal of research has been performed on the migration of water and its influence

on the stress-strain relationships (Whitman, 1960), however, definite ideas with respect to this fact have yet to be formed. It may have been in error to select the central portion but it was felt that in the majority of cases, this is where failure occurred and therefore would present the most reliable value.

A specific gravity test was performed on each triaxial sample, and the average specific gravity of the three samples from one Mohr envelope was used in calculations for these same three samples. The final volume of the triaxial specimen was obtained by mercury immersion. Error may occur in this method due to the entrapped air, flexibility of the plastic plate, or the penetration of the specimen by the prongs on the plate. These errors, along with inherent errors in the specific gravity due to salt content, may cause some deviation in values obtained for; degree of saturation, void ratio, porosity, and density.

4:6 Back Pressure

Due to the difficulty in producing a perfectly saturated sample, a system referred to as back pressuring was used in the triaxial testing of this work. Back pressuring consists of an application of a pressure in the sample concurrently with an application of an equal pressure in the triaxial chamber. In this manner the resultant intergranular pressure is still equal to the consolidation pressure. The increased neutral pressure in the soil sample will tend to dissolve the air in the water thereby providing more reliable pore pressure measurements. The strength of a soil varies with the degree of saturation, therefore if all samples are brought as close as possible to 100 percent saturation, better fitting Mohr envelopes will ensue. Complete saturation is required in

order that the condition of no volume change may be relied upon in undrained tests.

In driving the air into solution, the sample experiences a slight decrease in volume, thereby incorporating an affinity for water. In this testing programme the sample was allowed to take on or expel water in order to reach equilibrium. No difficulties arose during the testing procedure to indicate that the system was in error.

It may be advantageous to use the system of back pressuring outlined by Eden (1960). In this procedure back pressuring is carried out during the consolidation process. This method provides higher degrees of saturation.¹

4:7 Rate of Strain

The shear strength of a normally consolidated clay is dependent upon the rate of strain; Bishop and Henkel (1962, p.19), Whitman (1960, p.597). A rate of strain of 2% per hour was arbitrarily selected in order that the data obtained might be correlated with previous research (Thomson, 1963) and that failure might be reached in a normal working day. The rate of strain was maintained constant to prevent introducing another variable.

Bishop and Henkel (1962, p.192) state:

"The rate of testing has a marked influence in clay soils in particular, on the pore pressures observed during the undrained stage of the triaxial test. This arises from three causes:

- (a) time lag in the pore pressure device
- (b) progressive equalization of non-uniformity in pore

¹From conversations with C. Crawford, Soil Branch, Division of Building Research, National Research Council

- pressure resulting from a natural tendency to zone failure, and
- (c) a modification in the behavior of the soil structure as the rate of shear is reduced.

In choosing a rate of testing, it is necessary to be able to determine the relative influence of these three factors on the observed values of pore pressure, so that the results obtained will lead to a true measure of the physical properties of the soil."

To account for the above, the following factors were considered.

The time lag has been reduced considerably by the use of internal drains which improve dissipation of pore pressure, filter paper side drains which can accelerate the pore pressure response 100 times (Bishop and Henkel, 1962, p.193), back pressure which aids in dissolving the air, and careful preparation of a nearly saturated specimen. The filter strips and wool wicks which are in contact with the ceramic disc aid in equalizing the pore pressure throughout the sample (Bishop and Henkel, 1962, p.195). These aids should allow the chosen rate of strain. The special test which is described in the following section also substantiates the rate of strain used.

4:8 Special Triaxial Test With a Slow Rate of Strain

A special triaxial test was performed on the sodium-soil with zero salts in the pore water and a confining pressure of 5.0 Kg/cm^2 . The test was performed in the same manner as the other triaxial tests except that a rate of strain of 0.0475% per hour was used. The sample required approximately eight days to reach failure. When compared with an identical sample¹ which was performed at the rate of 2% per hour

¹ Sample No. 1-2, sodium-clay, zero salts in the pore water, confining pressure = 5.0 Kg/cm^2

the following factors were noted. The deviator stress varied by 0.10 Kg/cm^2 at failure which is a negligible amount. The pore pressure in the special test was 1.31 Kg/cm^2 at failure while in the standard test it was 0.57 Kg/cm^2 at failure. However, " \bar{A} " in the special test was 63.3% which compares favorably with the majority of triaxial tests performed on sodium-clay with zero salts in the pore water (Thomson, 1963, Appendix I). Also the Mohr envelope for zero salt content shows that the Mohr circle from the standard test run at 5.0 Kg/cm^2 confining pressure did not fit favorably, but if it had had a slightly higher pore pressure the Mohr circle would have fit better. Therefore, it may be said that the deviation in pore pressure was not significant as the results appear to indicate because the pore pressure in the standard test was too low due to unknown circumstances.

This test indicated that the rate of strain employed was favorable to develop the correct stress and pore pressure measurements.

The question of possible strength variation due to thixotropy developing due to storage maybe answered by this test. The sample tested had been stored for approximately five months. The slight variation in results indicates that it may be assumed that thixotropy has no significant influence.

4:9 Pore Pressure Reaction Test

The response of the pore pressure measuring system was checked by increasing the chamber pressure without allowing the sample to drain and observing the pore pressure response. In Table IV the value of " B "* indicates the response in pore pressure (within two minutes) to an increase

in the cell pressure of 1 Kg/cm^2 . In the majority of cases the value of "B" was in the range of 80% to 95% which indicates a relatively well saturated specimen. Some of the low values were due to difficulties with the apparatus and these have been indicated in the table by "X".

4:10 Failure Criteria

The failure criteria generally adopted in triaxial testing of cohesive soils are the maximum deviator stress and the maximum principal effective stress ratio. Bjerrum and Simons (1960, p.714) state: "The pore pressure at the point of maximum deviator stress either attains a maximum value and the two failure criteria coincide, or it is still increasing and only reaches a maximum value upon further strain. If the deviator stress is constant or only decreases slightly with strain after its maximum value, then the maximum principal effective stress ratio point occurs after the point of maximum deviator stress."

The value of the angle of shearing resistance will be higher for the latter case. Normally with the triaxial tests run in this work, the pore pressure increased slightly with further strain past the point of maximum deviator stress. Due to the fact that the points of maximum deviator stress and maximum principal stress ratio were nearly coincident, the criterion chosen was not critical to the final results. The maximum principal effective stress ratio was adopted in this investigation and all Mohr circles were defined on this basis.

4:11 Mohr Envelopes

Triaxial tests were performed at minor total principal stress values of 2.5 Kg/cm^2 , 5.0 Kg/cm^2 and 7.5 Kg/cm^2 . The values were chosen to

provide a uniform distribution of circles so that the best Mohr envelope possible could be attained. Only three circles were used to define an envelope as the periods of time involved with each test ranged from one week to two months. Therefore, a minimum number of circles were obtained in order that the testing programme might be completed within a reasonable length of time.

In order to eliminate personal bias, the Mohr envelopes were fitted by the method of least squares. In nearly every case the rupture line fits the circles satisfactorily.

4:12 Flame Photometer

The total exchange capacity, the type and percentage of cation adsorbed, and the concentration of salts in the pore water were determined by the Beckman Flame Photometer and titration procedure. With the flame photometer, a value for the total salt content was determined by finding the amounts of calcium, potassium, sodium and magnesium present. The titration procedure provides a value for the total exchange capacity. The concentration of salts in the pore water was taken as the difference between the total exchange capacity and the total salt content.

The general procedure employed was that in use by the Alberta Research Council, Soil Survey Section. The only variation from this procedure was that five grams of soil passing a No. 40 sieve were used instead of 20 grams of a coarser size.

The error incorporated in the flame photometer determinations is in the order of 1-3%¹. Titration error is not known but may be assumed to be of the order of 4-6%

¹ From conversations with Dr. S. Pawluk, Department of Soil Science University of Alberta

CHAPTER V

PRESENTATION OF TEST RESULTS

5:1 General

The problem of this thesis was to investigate the variation in shear strength with a change in the concentration of salts in the pore water for two homionic soil modifications. The shear strength may be defined by Mohr envelopes, either in terms of effective or total stress, from the results of consolidated quick triaxial tests with pore pressure measurements. In this chapter only the rupture lines are shown for simplification in presenting results. The values for the angles of shearing resistance are given in degrees and minutes as obtained by treating the data by the method of least squares. Mohr envelopes with their respective circles are in Appendix A. In this thesis the terms concentration, salt content, and electrolyte concentration refer to the concentration of salts in the pore water expressed in units of me/100 gms. a.d.s. The results of the testing programme are presented in this chapter; the discussion of the results are presented in the following two chapters.

5:2 Shear Strength of the Sodium Modification

Figure 1 shows the Mohr envelopes for the sodium-clay modification with varying concentrations of salts in the pore water. Table III lists the values of cohesion^{*} and angles of shearing resistance for this soil.

The sodium modification shows a variation in the effective angle of shearing resistance from $12^{\circ}43'$ for zero salts in the pore water to a

TABLE III

ANGLES OF SHEARING RESISTANCE AND COHESION

FOR

SODIUM MODIFICATION

Salt Content	Angles		Cohesion	
me/100 gms. a.d.s.	Effective	Total	Effective	Total
0	12°43'	9°46'	-0.11	0.00
2.3	13°05'	9°21'	0.27	0.26
4.7	14°57'	10°10'	0.22	0.31
7.6	25°33'	13°16'	-0.06	0.11
13.1	23°57'	13°10'	0.15	0.11

maximum of $25^{\circ}33'$ for 7.6 me/100 gms. a.d.s. and then a slight decrease to $23^{\circ}57'$ for 13.1 me/100 gms. a.d.s. In terms of the total angle of shearing resistance, the values generally increased from $9^{\circ}46'$ with zero salts in the pore water to a maximum of $13^{\circ}16'$ with 7.6 me/100 gms. a.d.s. and then decreased slightly to $13^{\circ}10'$ for 13.1 me/100 gms. a.d.s. With respect to cohesion the variation ranges from zero to 0.31 Kg/cm^2 and this variation shows a slight tendency to be similar to the angle of shearing resistance. With such low values the cohesion parameter tends to be sensitive to fluctuations when fitting the Mohr envelope. Therefore, the prementioned trend is not conclusive but may be of some significance. The author has chosen therefore, to refrain from comparing actual cohesion values.

The values of the angles of shearing resistance for the sodium-clay modification are presented graphically by plotting them against salt content in the pore water, Figure 2. The proposed relationships are represented by dashed lines as the number of points available is considered insufficient to adequately express the variation in the angle of shearing resistance for all salt contents.

The total angle of shearing resistance decrease slightly from zero salt to 2.3 me/100 gms., a.d.s. rises gently to 4.7 me/100 gms. a.d.s., rises sharply to 7.5 me/100 gms. a.d.s., and then levels off. The effective angle of shearing resistance shows a slight increase to 2.3 me/100 gms. a.d.s., a sharper rise to 4.7 me/100 gms. a.d.s., an abrupt increase to 7.6 me/100 gms. a.d.s., and then a slight decrease to 13.1 me/100 gms. a.d.s. The variations are more pronounced with the effective angle of shearing resistance than with the total angle of shearing resistance. The curves

suggest that a maximum angle of shearing resistance exists at approximately 8 me/100 gms. a.d.s.

A plot of the logarithm of compressive strength versus moisture content for saturated clays is generally considered to be a unique curve (Taylor, 1958, p.371). A definite linear relationship is shown to exist between these two factors for each concentration of salts in the pore water, Figure 3. The slope of the curves decreases as the concentration of salts in the pore water increases up to at least 7.6 me/100 gms. a.d.s, after which the curve for 13.1 me/100 gms. a.d.s. becomes slightly steeper once again. This trend coincides directly with the variations in the angles of shearing resistance. There is also a downshift of the curves with an increase in salt content.

Figure 4 to 7 were taken from triaxial tests at each salt content performed with a confining pressure of 5.0 Kg/cm^2 . They are typical plots of the shown relationships. Each factor is plotted versus percent strain. The deviator stress, Figure 4, shows a definite increase in the maximum value with an increase in salt content. This plot may be considered to be representative of the major total stress also, as the value of the minor total stress is constant for these curves. The nature of the curve at low salt contents is in general similar to those of higher salt contents. The significant difference with low salt concentrations is that the deviator stress dropped off quickly after reaching its maximum value at a relatively low strain, approximately 3-5%. At higher salt contents the maximum deviator stress occurred at higher strains, approximately 8-10% and then decreased very gradually.

Figure 5 illustrates the increase in pore pressure with the increase in salt content. The dashed (assumed) curve for zero salts in the pore water was taken from the special test at the low rate of strain (Sec. 4:3). The actual curve for the recorded pore pressure for zero salt in the pore water ($\sigma_3 = 5.0 \text{ Kg/cm}^2$) was assumed to be low due to the following indications: (1) the pore pressure parameter \bar{A} was low for the original test when compared to similar tests (Thomson, 1963, Appendix I), (2) the effective Mohr circle for the original test would have produced a better envelope with a higher pore pressure. In any case it is evident that lower values of pore pressure are developed at lower salt contents. This fact is substantiated by the average pore pressure parameter \bar{A} which generally increased with salt content except for the values at higher salt contents which fluctuated at approximately 83% (Note Table IV). Also the pore pressure continued to increase after failure occurred for the low salt contents, but for high salt contents the pore pressure tended to level off after failure.

Figure 6 shows the slight variation in the major principal effective stress (σ_1') with electrolyte concentration. With high salt concentrations it may generally be said that σ_1' increases to its maximum value at lower strains than it does at lower salt content. When studying this factor, if one also studies Figure 4 and 5 it appears that the major total stress increases at a greater rate than the increase in pore pressure at low strains with an increase in salt content. This factor accounts for the nature of the variation in the major principal effective stress.

TABLE IV
SUMMARY OF TEST DATA

Sample		Final M.C.	Void Ratio		Deg. of Sat.		Consolidation		Triaxial							Classification				Cation Determinations						Shear Strength																																			
No.	Desc.	%	Init.	Fin.	Init.	Fin.	t ₁₀₀	t ₉₀	σ ₃	σ ₁ - σ ₃	P _p	σ ₁	% Strain	σ ₁ '	σ ₃ '	Ā	B	pH	G _s	W _L	P _L	CEC	TEC	SC	%K ⁺	%Ca ⁺⁺	%Na ⁺	φ	φ	C'	C																														
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32																														
SODIUM MODIFICATION																																																													
1-1	0.0 N	64.04	2.52	1.92	97.26	95.61	58,000	50,000	2.5	1.05	0.32	3.55	4.5	3.23	2.18	30.5	56	7.7	2.85	--	--	--	30.4	33.3	0.0	1.3	0.0	97.7																																	
1-2	0.0 N	53.14	2.36	1.57	100.73	96.97	54,000	48,000	5.0	1.97	0.57	6.97	4.0	6.40	4.43	28.9	63	7.8	2.87	--	--	--	30.9	33.7	0.0	2.2	1.5	96.3																																	
1-4	0.0 N	46.55	2.38	1.39	98.94	95.94	57,000	48,000	7.5	3.09	1.79	10.69	3.75	8.80	5.71	41.5	70	7.4	2.86	--	--	--	32.6	34.7	0.0	2.0	6.5	91.5																																	
Avge. Data							56,300	48,600								33.6			2.86	92.8	31.1	31.3	33.9	0.0	1.8	2.6	95.2	12°43'	9°46'	-0.11	0.00																														
2-1	0.1 N	40.47	2.00	1.19	96.67	95.72	15,600	9,600	7.5	3.50	2.57	11.00	9.0	8.43	4.93	73.4	30*	7.5	2.80	--	--	--	32.9	33.1	0.0	1.3	0.0	98.7																																	
2-2	0.1 N	52.14	1.98	1.52	98.24	96.53	24,000	19,600	2.5	1.56	0.86	4.06	4.5	3.20	1.39	55.1	96	7.4	2.84	--	--	--	34.2	32.7	1.5	1.2	3.9	94.9																																	
2-3	0.1 N	44.42	1.97	1.29	96.72	96.94	26,000	14,600	5.0	2.62	1.95	7.62	7.5	5.67	3.05	74.4	90	7.3	2.82	--	--	--	38.7	33.2	5.5	2.4	1.8	79.4																																	
Avge. Data							22,000	14,600								67.3			2.83	78.0	31.4	35.3	33.0	2.3	1.6	1.9	91.0	13°05'	9°21'	0.27	0.26																														
5-1	0.2 N	37.85	1.82	1.12	95.80	95.44	9,800	4,600	7.5	3.71	3.32	11.21	7.5	7.89	4.18	89.5	20*	--	2.84	--	--	--	37.2	32.9	4.3	0.7	0.0	99.3																																	
5-2	0.2 N	41.04	1.82	1.21	96.35	95.80	5,200	4,100	5.0	2.73	2.26	7.73	11.5	5.47	2.74	82.8	30*	7.5	2.81	--	--	--	36.3	30.3	6.0	1.2	0.0	98.8																																	
5-3	0.2 N	47.85	1.82	1.40	96.63	96.62	4,700	2,900	2.5	1.57	1.40	4.07	9.0	2.67	1.10	89.2	80	--	2.82	--	--	--	36.7	33.0	3.7	0.6	0.0	99.4																																	
Avge. Data							6,600	3,900								87.2			2.82	71.5	31.3	36.7	32.0	4.7	0.8	0.0	99.2	14°57'	10°10'	0.22	0.31																														
3-1R	0.3 N	46.08	1.73	1.33	99.60	97.26	620	320	2.5	1.75	1.18	4.25	8.0	1.32	3.07	67.4	86	7.6	2.80	--	--	--	42.4	35.1	7.3	1.8	0.0	98.2																																	
3-2	0.3 N	37.89	1.79	1.11	97.79	96.05	980	600	7.5	4.69	4.24	12.19	11.0	7.95	3.26	90.4	42	7.5	2.81	--	--	--	39.6	34.1	5.5	1.2	0.0	98.8																																	
3-3	0.3 N	40.87	1.82	1.18	97.85	97.02	740	500	5.0	3.38	2.73	8.38	11.0	5.65	2.27	80.8	64	7.1	2.81	--	--	--	44.4	34.3	10.1	1.3	0.0	98.7																																	
Avge. Data							780	475								79.5			2.81	71.7	32.7	42.1	34.5	7.6	1.4	0.0	98.6	25°33'	13°16'	-0.06	0.11																														
4-1	0.5 N	36.34	1.80	1.06	91.11	96.17	400	320	7.5	4.74	4.29	12.24	11.5	7.95	3.21	90.5	67	--	2.80	--	--	--	43.2	33.5	9.7	1.5	0.0	98.5																																	
4-2	0.5 N	38.93	1.80	1.13	90.28	96.04	290	400	5.0	3.46	2.71	8.46	10.5	5.75	2.29	78.3	86	7.1	2.82	--	--	--	48.4	33.3	15.1	0.8	0.0	99.2																																	
4-3	0.5 N	44.72	1.80	1.30	90.27	96.48	210	120	2.5	1.80	1.44	4.30	8.0	2.86	1.06	80.0	90	--	2.79	--	--	--	47.9	33.5	14.4	0.7	0.0	99.3																																	
Avge. Data							335	240								82.9			2.80	69.8	31.8	46.5	33.4	13.1	1.0	0.0	99.0	23°57'	13°10'	0.15	0.11																														
CALCIUM MODIFICATION																																																													
A-1	0.0 N	36.16	1.72	1.01	96.31	98.38	170		7.5	4.71	4.29	12.21	13.5	12.21	3.21	91.1	84	7.0	2.77	--	--	--	45.8	33.7	12.1	0.9	98.8	0.3																																	
A-2	0.0 N	39.53	1.72	1.12	97.32	97.24	125		5.0	3.45	2.84	8.45	11.0	8.45	2.16	83.2	44	--	2.75	--	--	--	47.7	34.2	13.5	1.1	98.1	0.8																																	
A-3	0.0 N	45.58	1.68	1.28	97.20	97.59	175		2.5	1.74	1.59	4.24	12.5	4.24	0.91	91.4	92	--	2.74	--	--	--	47.2	33.7	13.5	1.0	98.8	0.2																																	
Avge. Data							156									88.6			2.75	78.8	31.7	46.9	33.9	13.0	1.0	98.6	0.4	23°08'	13°15'	0.19	0.13																														
B-2	0.3 N	38.82	1.72	1.12	95.82	96.38	185		5.0	3.26	2.95	8.26	14.5	8.26	2.05	90.2	89	--	2.74	--	--	--	51.4	33.6	17.8	1.1	96.8	2.1																																	
B-3	0.3 N	46.21	1.67	1.32	97.58	97.07	130		2.5	1.82	1.50	4.32	10.0	4.32	1.00	82.4	97	7.5	2.76	--	--	--	55.7	32.0	23.7	0.3	99.4	0.3																																	
B-4	0.3 N	35.53	1.75	1.02	94.48	96.59	130		7.5	4.77	4.21	12.27	9.5	12.27	3.29	88.3	56	--	2.77	--	--	--	48.1	32.5	15.2	0.8	98.6	0.6																																	
Avge. Data							148									86.9			2.76	76.5	30.5	51.7	32.8	18.9	0.7	98.3	1.0	23°02'	13°10'	0.19	0.13																														
C-1	0.8 N	33.76	1.80	1.00	89.24	94.96	230		7.5	4.61	4.36	12.11	12.0	12.11	3.14	94.6	93	--	2.73	--	--	--	92.1	31.5	60.6	0.3	99.3	0.4																																	
C-2	0.8 N	35.92	1.74	1.02	90.15	96.60	190		5.0	3.10	2.82	8.10	8.5	8.10	2.18	91.0	92	7.6	2.75	--	--	--	97.8	32.1	65.7	0.6	98.9	0.5																																	
C-3	0.8 N	40.55	1.74	1.14	87.48	97.04	270		2.5	1.79	1.50	4.29	9.5	4.29	1.00	82.8	96	--	2.75	--	--	--	110.4	32.0	78.4	0.7	99.1	0.2																																	
Avge. Data							230									89.5			2.74	66.0	30.7	100.1	31.8	68.3	0.5	99.1	0.4	23°22'	12°43'	0.13	0.14																														
NATURAL WASHED CLAY																																																													
1		38.58	1.55	1.11	95.21	97.24	200		2.75	1.83	1.68	4.58	12.5	2.90	1.07	91.8	94	--	--	--	--	--	75.7	24.4	51.1	1.5	88.3	0.8																																	
2		33.68	1.48	0.97	91.15	97.30	200		5.25	3.05	2.82	8.30	10.0	5.47	2.42	92.8	77	--	--	--	--	--	80.6	28.4	52.5	1.2	88.0	0.9																																	
3		31.60	1.55	0.92	96.39	96.54	360		7.50	3.95	4.32	11.45	10.0	7.13	3.18	109.0	85	--	--	--	--	--	91.7	28.2	63.5	1.3	88.4	0.8																																	
Avge. Data							250									97.8			2.80	--	--	--	82.7	27.0	55.7	1.3	88.2	0.8	18°34'	10°32'	0.32	0.26																													

6, 7 Initial and Final Degree of Saturation, %
8, 9 Consolidation times, minutes
10-13, 15, 16 Values in kg/cm²
17, 18 percent

CEC Cation Exchange Capacity, me/100 gms., a.d.s.
TEC Total Exchange Capacity, me/100 gms., a.d.s.
SC Salt Content, me/100 gms., a.d.s.

The plot of minor principal effective stress versus percent strain, Figure 7, was considered to be similar to that of the pore pressure, Figure 6, as the minor total principal stress is a constant for these tests.

The average values of the pore pressure parameter \bar{A} are higher at higher salt contents, Table IV, i.e., the increase in pore pressure is greater than that of the deviator stress at higher salt contents.

5:3 Shear Strength of the Calcium Modification

Shear strength values for the calcium-soil did not vary significantly within the range of the salt contents studied, Figure 8. Concentrations of salts in the pore water used were 13.0, 18.9 and 68.3 me/100 gms. a.d.s. For the respective salt contents the values for the total angle of shearing resistance were $13^{\circ}15'$, $13^{\circ}10'$ and $12^{\circ}43'$ and for the effective angle of shearing resistance the values were $23^{\circ}08'$, $23^{\circ}02'$ and $23^{\circ}22'$. Note Table V. Both the total and the effective angles of shearing resistance were higher than the respective values determined for the natural washed clay. The natural washed clay had 88.7% of its positions filled with calcium and had a salt content of 55.6 me/100 gms. a.d.s.

TABLE V
ANGLES OF SHEARING RESISTANCE AND COHESION FOR
CALCIUM MODIFICATION

me/100 gms. a.d.s				
	Effective	Total	Effective	Total
13.9	$23^{\circ}08'$	$13^{\circ}15'$	0.19	0.13
18.9	$23^{\circ}02'$	$13^{\circ}10'$	0.19	0.13
68.3	$23^{\circ}22'$	$12^{\circ}43'$	0.13	0.14

The plot of moisture content versus the logarithm of compressive strength, Figure 9, shows that for the high salt content, 68.3 me/100 gms. a.d.s., the moisture contents were on the average about 7% lower for a comparable strength than that of lower salt contents. The values corresponding to the other two salt contents nearly coincide. The slope of the curve did not vary a significant amount with change in salt content. The curves do, however, tend to shift downward with an increase in salt content.

5:4 Triaxial Consolidation With Sodium and Calcium Modifications

Time curves were drawn using the logarithm of time fitting method (Figure 10) and the square root of time fitting method, (Figure 11), (Taylor, 1948, p.241). From these plots t_{100}^* and t_{90}^* may be determined. Figure 10 shows the logarithmic time curves for the sodium-clay modification. The general shape of the curves is very similar for all salt contents but the curves shift to the right thereby illustrating large increases in t_{100} as the salt content decreases. Sodium-clay with zero salts in the pore water has an average t_{100} of 56,300 minutes, (about six weeks). With zero salts in the pore water of a sodium-clay, the lengths of time required to reach t_{100} are so large that it is difficult to clearly distinguish when the curve begins to bend at theoretical 100% consolidation. However, when the time curves were drawn with the square root of time plotted on an arithmetic scale, there was a definite break in the curve denoting that secondary compression had been reached. Therefore the time curves were fitted by the logarithm of time method and the square root of time method. If t_{100} and t_{90} are plotted on a logarithmic scale versus salt content, Figure 12, the variation in time to reach secondary compression with respect to the salt content can be readily noted. The curve falls rapidly to a salt

content of approximately 7 me/100 gms. a.d.s. where it appears to curve sharply, and one would presume the curve to become horizontal.

With the calcium-clay modification variations in t_{100} are slight; however, there appears to be a slight increase in t_{100} at the high salt content as illustrated in Figure 13. The curve has been presented as a dashed line joining the points due to the limited amount of data.

5:5 Atterberg Limits for Sodium and Calcium Modifications

All calculations for liquid and plastic limits include the weight of salts with the weight of oven dried soil. Figure 14 shows the liquid and plastic limits versus salt content. Salt contents are based on the flame photometer results obtained for the triaxial tests.

Figure 14 shows the decrease in liquid limit with an increase in salt content for the sodium clay modification (Note Table IV). The liquid limit decreases from 92.8 at zero salts in the pore water to 71.1 at 7.6 me/100 gms. a.d.s. and then tends to level off with a further increase in salt content. The plastic limit for the sodium-clay tends to stay constant at approximately 31.6. Any variations from this value are within range of experimental error.

The liquid and plastic limits for the calcium-clay modification are given in Table VI. The plastic limit remains relatively constant despite the change in salt content. There is, however, a decrease in the liquid limit with an increase in electrolyte concentration.

By plotting the liquid limit and plasticity index on the plasticity chart, Figure 14, a definite relationship appears. The points form a straight line which is at a slope to the A-line, and tends to approach the A-line with an increase in salt content. The values from both soil

modifications fall on the same line but a higher concentration of salts is required for the calcium-clay to obtain a point comparable to the sodium-clay with a lower salt content.

TABLE VI
LIQUID LIMITS AND PLASTIC LIMITS
FOR
CALCIUM MODIFICATIONS

Salt Content me/100 gms. a.d.s	Liquid Limit	Plastic Limit
13.0	78.8	31.7
18.9	76.5	30.5
68.3	66.0	30.7

5:6 Specific Gravity of Soil Modifications

Table VII lists the results of the specific gravity tests on the sodium-clay and the calcium-clay. From the results, the sodium clay shows a decrease in specific gravity with an increase in salt content. The specific gravity decreases when the adsorbed cation is calcium, but remains relatively constant despite the variation in salt content.

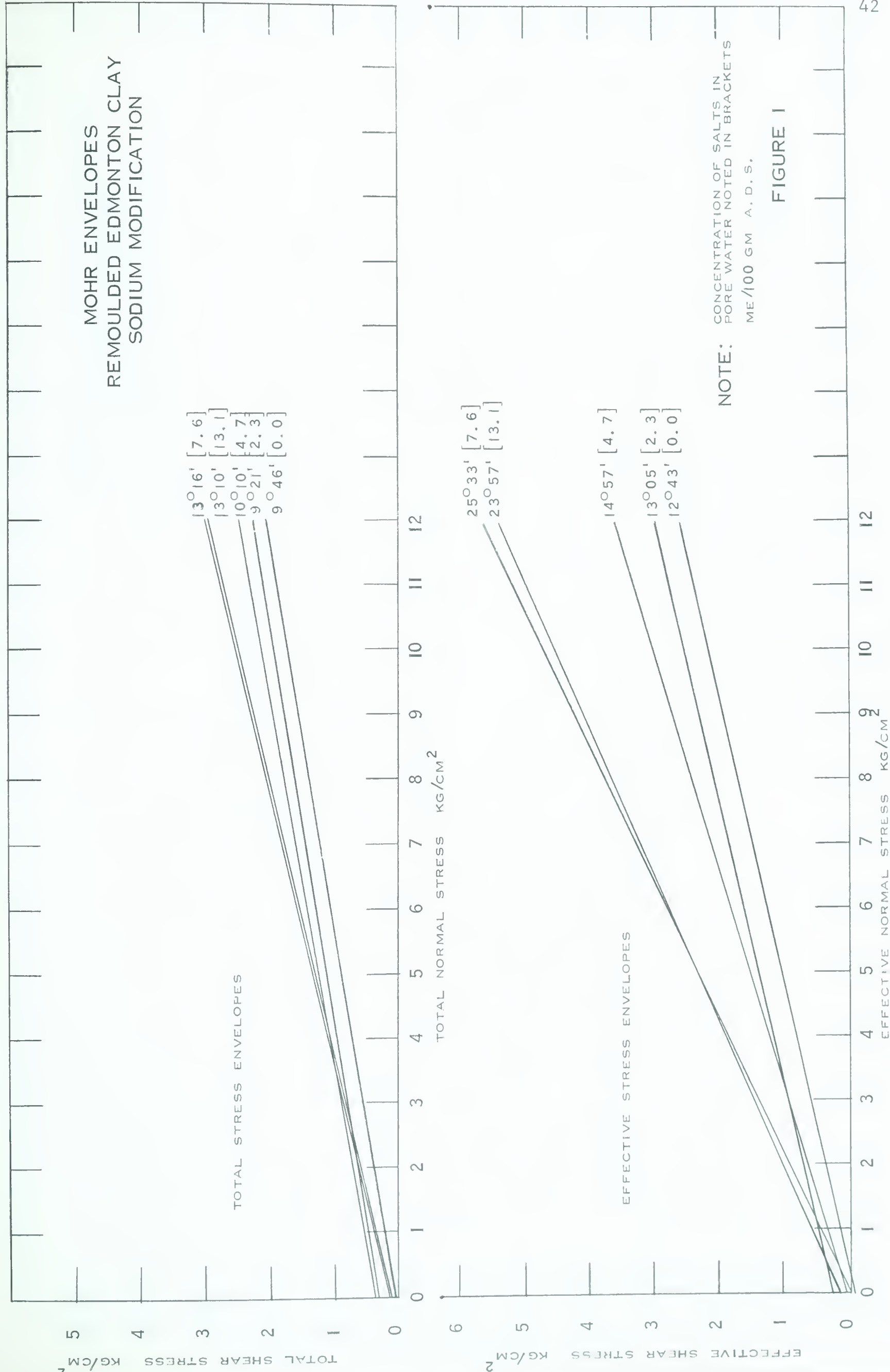
TABLE VII

SPECIFIC GRAVITY OF SOIL SOLIDS

FOR

SODIUM AND CALCIUM MODIFICATIONS

Soil Modification	Individual Determinations	Average Value	Soils in Pore Water me/100 gms. a.d.s.
Sodium	2.85, 2.86, 2.87	2.86	0
"	2.80, 2.82, 2.84	2.83	2.3
"	2.81, 2.82, 2.84	2.82	4.7
"	2.80, 2.81, 2.81	2.81	7.6
"	2.79, 2.80, 2.82	2.80	13.1
Calcium	2.74, 2.75, 2.77	2.75	13.0
"	2.74, 2.76, 2.77	2.76	18.9
"	2.73, 2.75, 2.75	2.74	68.3



ANGLES OF SHEARING RESISTANCE
VS SALT CONTENT
SODIUM MODIFICATION

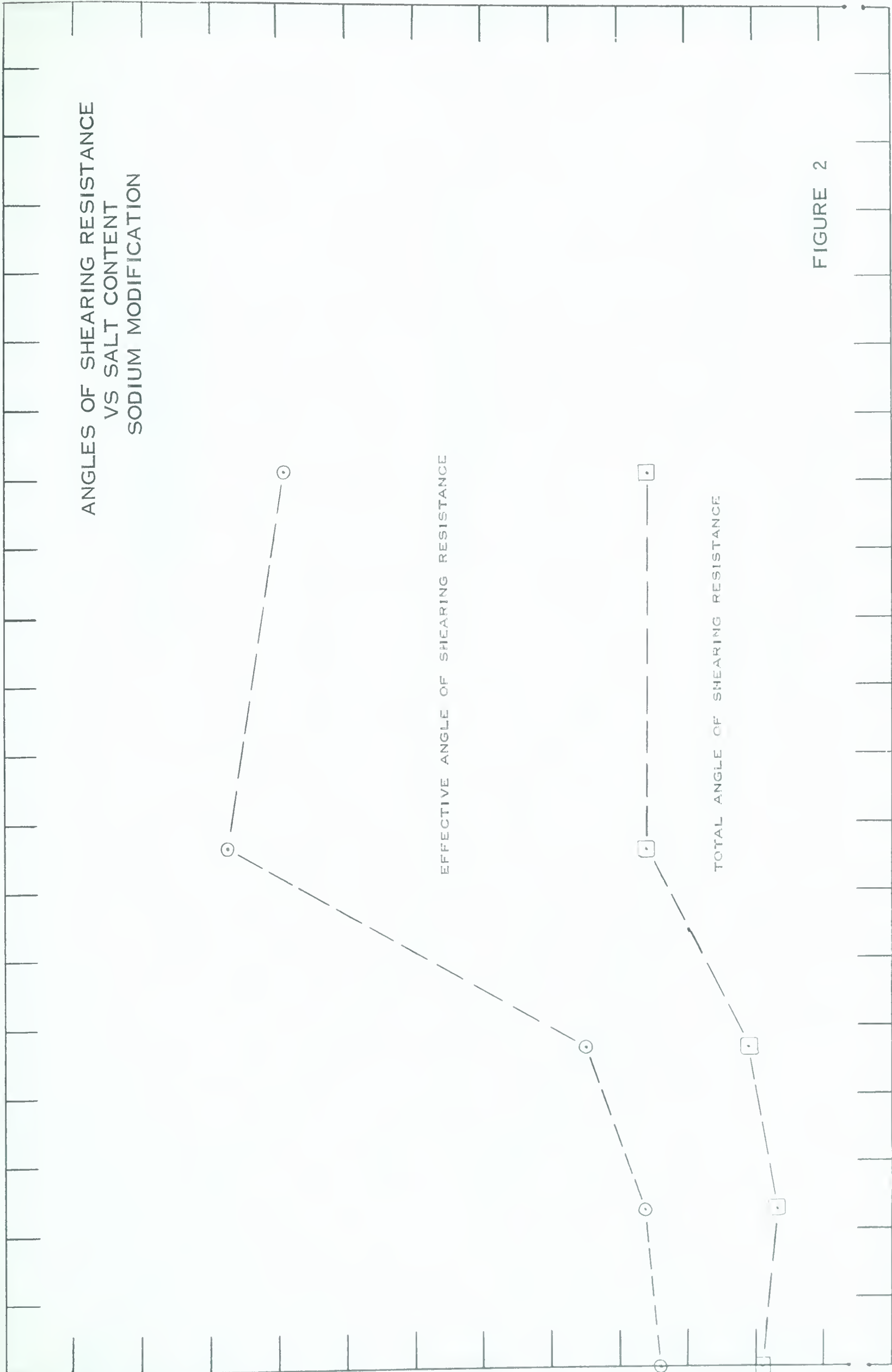
ANGLE OF SHEARING RESISTANCE [DEGREES]

EFFECTIVE ANGLE OF SHEARING RESISTANCE

TOTAL ANGLE OF SHEARING RESISTANCE

SALT CONTENT ME/100 GM A.I.S.

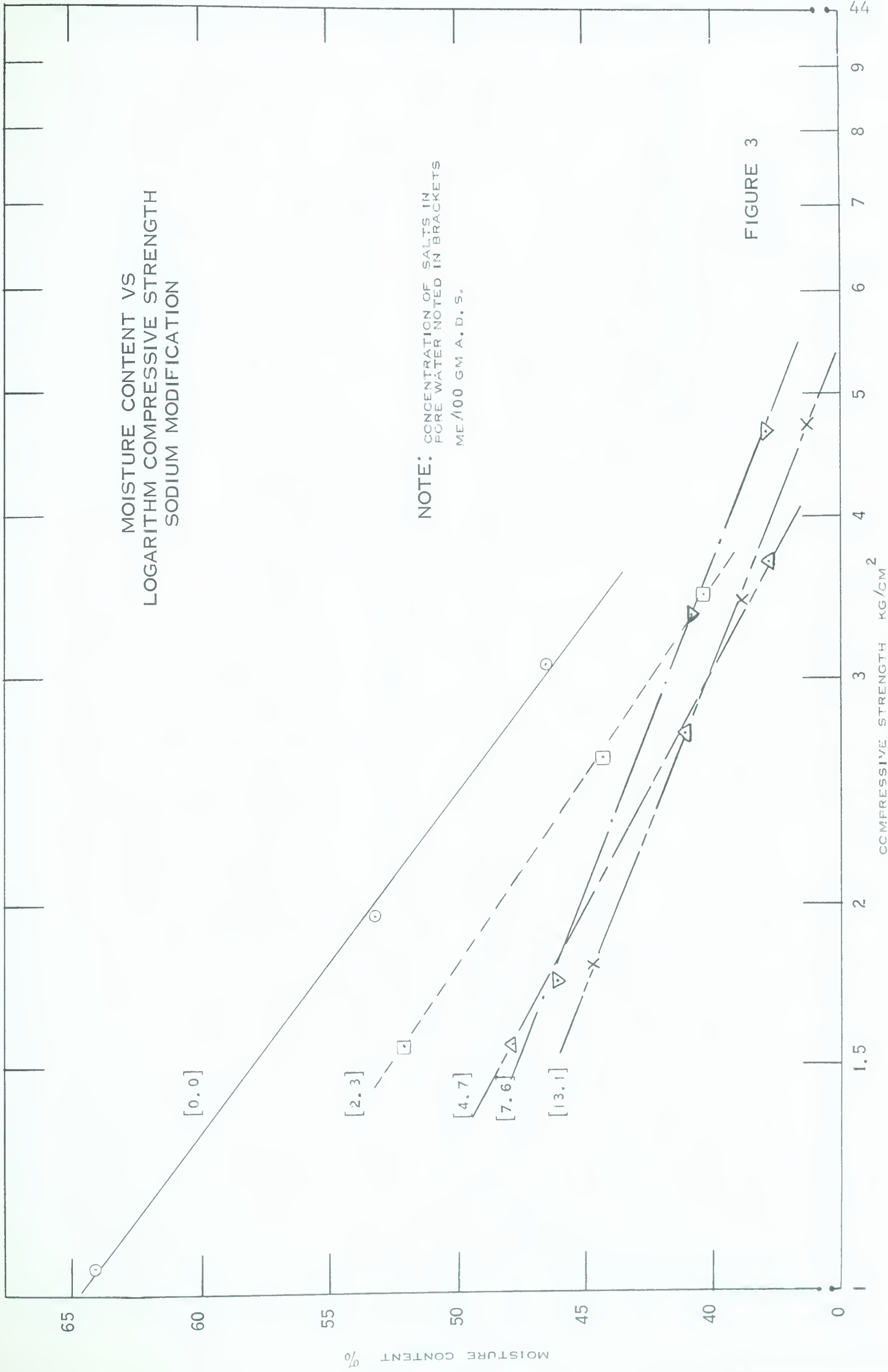
FIGURE 2



MOISTURE CONTENT VS LOGARITHM COMPRESSIVE STRENGTH SODIUM MODIFICATION

NOTE: CONCENTRATION OF SALTS IN
PORE WATER NOTED IN BRACKETS
ME/100 GM A.D. S.

FIGURE 3



DEVIATOR STRESS VS
AXIAL COMPRESSIVE STRAIN
SODIUM MODIFICATION
 $\sigma_3 = 5 \text{ kg/cm}^2$



NOTE: CONCENTRATION OF SALTS IN
PORE WATER NOTED IN BRACKETS
ME/100 GM A. D. S.

FIGURE 4

PORE PRESSURE VS
AXIAL COMPRESSIVE STRAIN
SODIUM MODIFICATION
 $\sigma_3 = 5 \text{ KG/CM}^2$

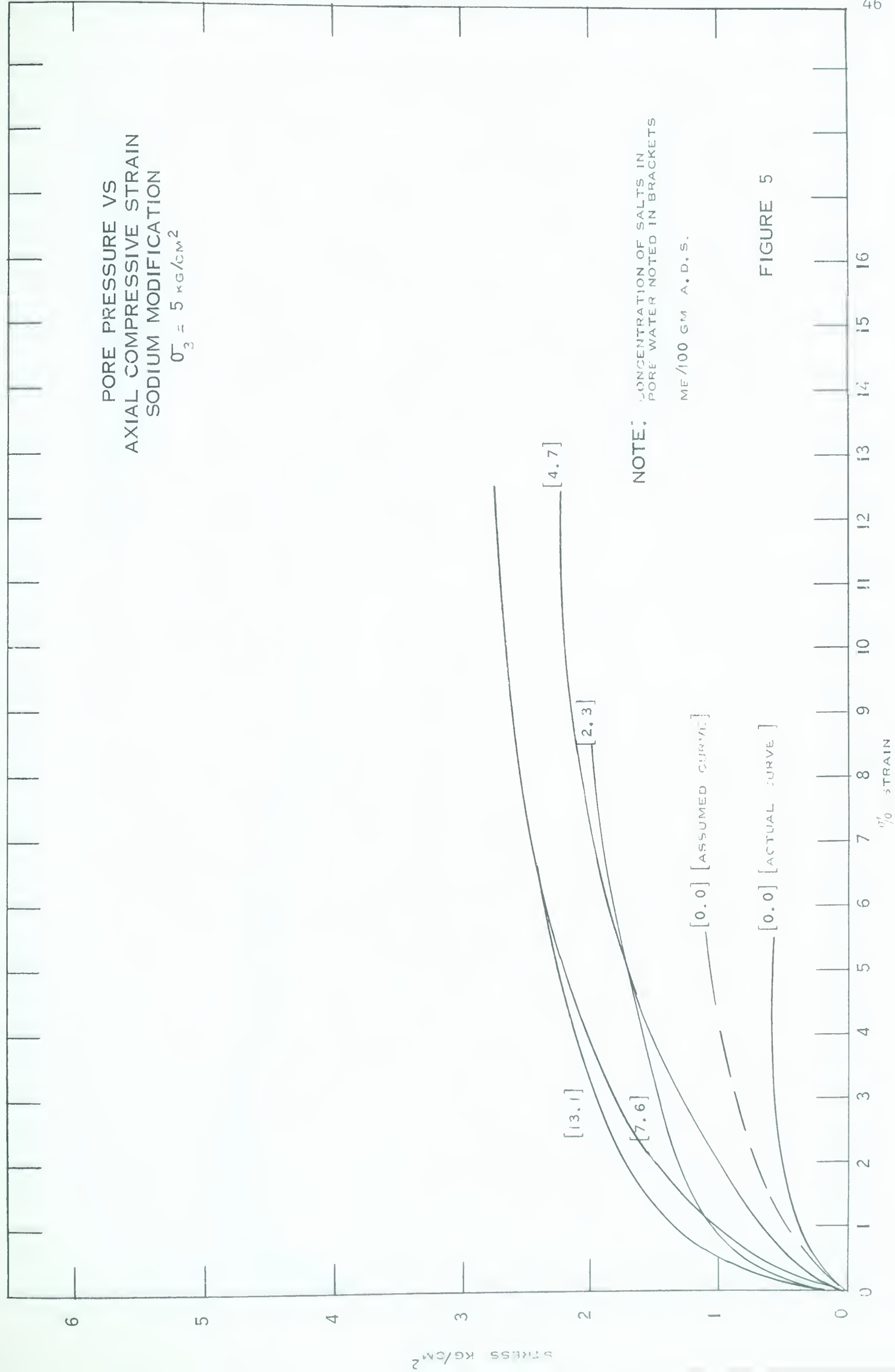
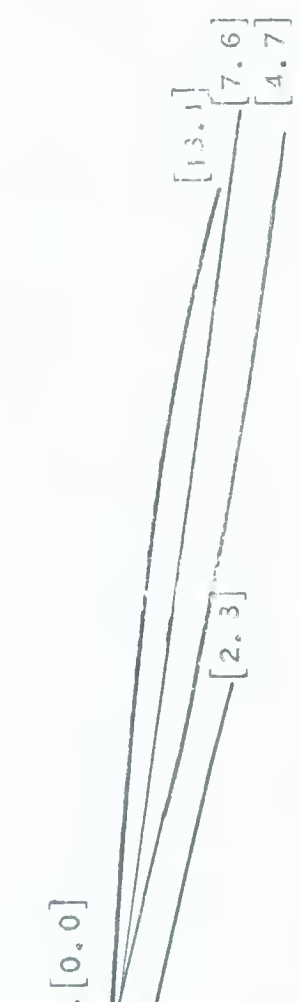


FIGURE 5

MAJOR PRINCIPAL EFFECTIVE STRESS
VS AXIAL COMPRESSIVE STRAIN
SODIUM MODIFICATION
 $\sigma_3 = 5 \text{ KG/CM}^2$



NOTE: CONCENTRATION OF SALTS IN
PORE WATER NOTED IN BRACKETS
ME/100 GM A.D.S.

FIGURE 6

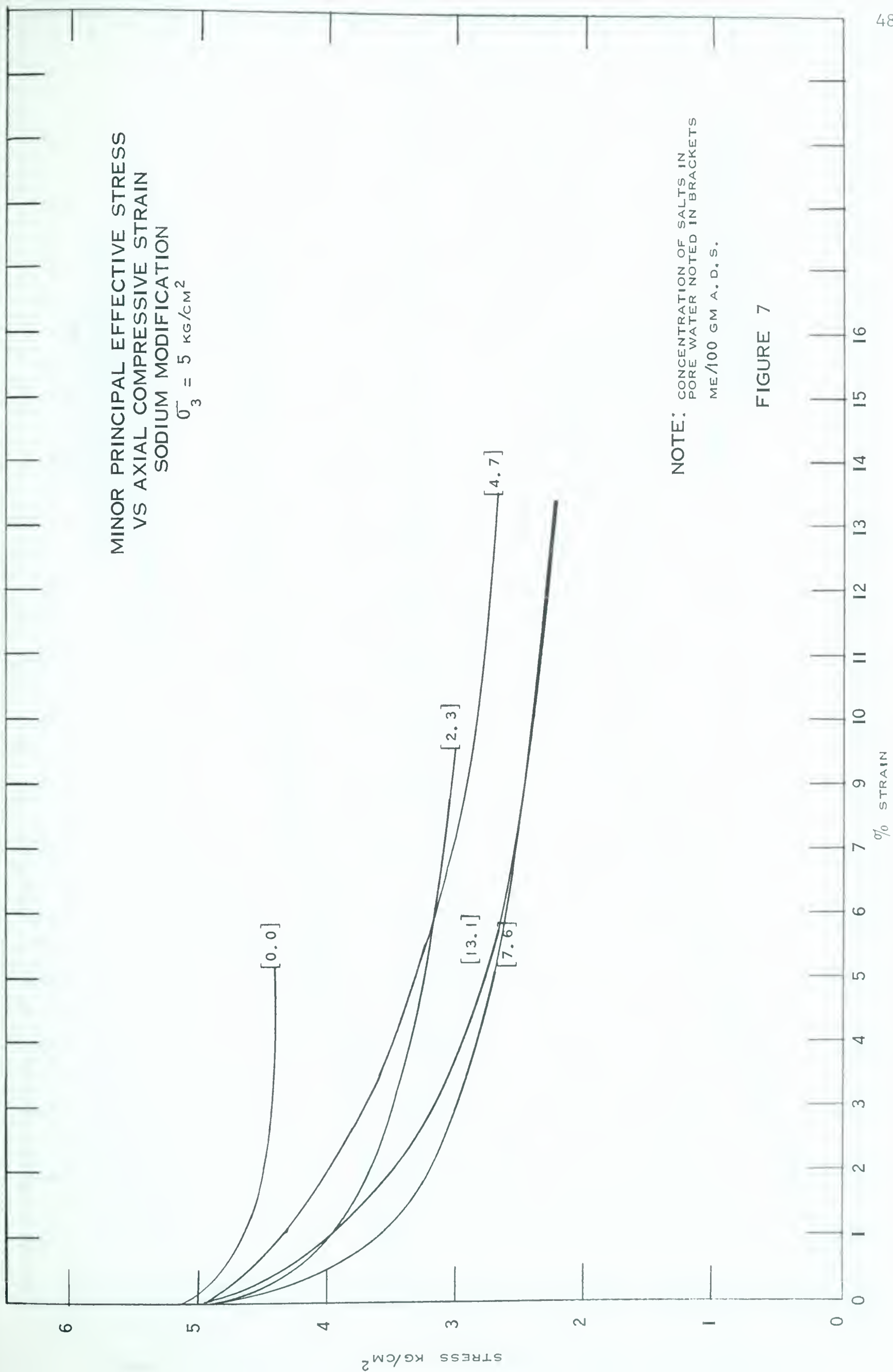


FIGURE 7

% STRAIN

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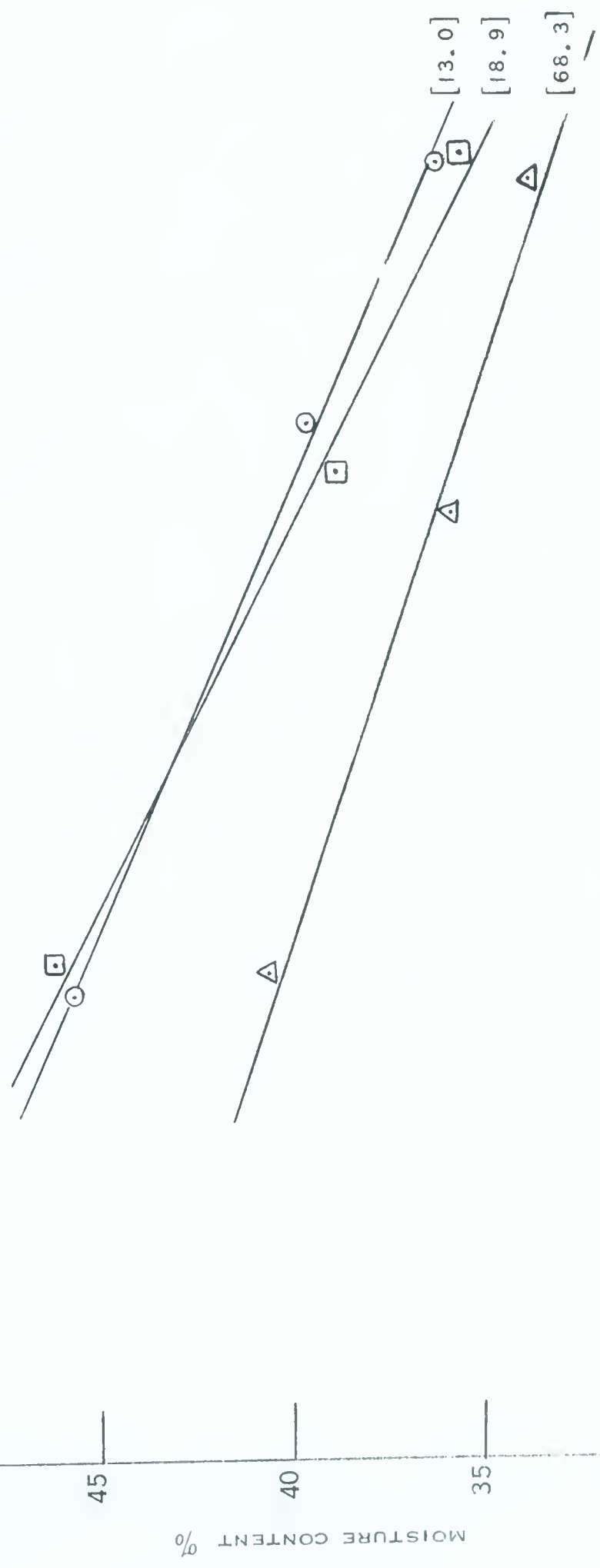
MOHR ENVELOPES
 REMOULDED EDMONTON CLAY
 CALCIUM MODIFICATION
 AS COMPARED TO NATURAL WASHED CLAY

AVERAGE 13'03" [13.0, 18.9, 68.3]
 10'32" NATURAL WASHED CLAY [55.7]

NOTE: CONCENTRATION OF SALTS IN
 PORE WATER NOTED IN BRACKETS
 ME/100 GM A.D.S.

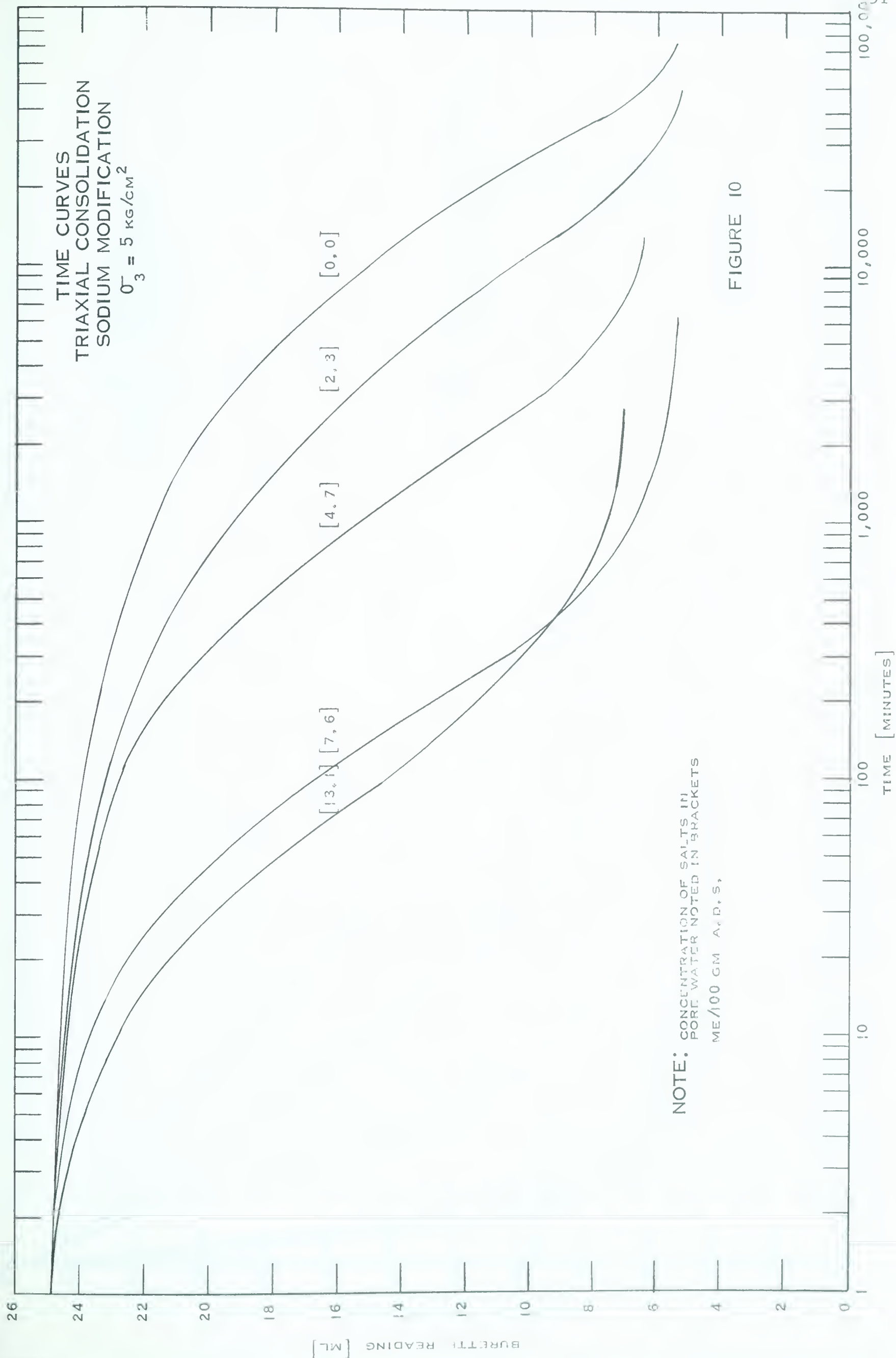
FIGURE 8

MOISTURE CONTENT VS
LOGARITHM COMPRESSIVE STRENGTH
CALCIUM MODIFICATION



NOTE: CONCENTRATION OF SALTS IN
PORE WATER NOTED IN BRACKETS
ME/100 GM A.L.S.

FIGURE 9



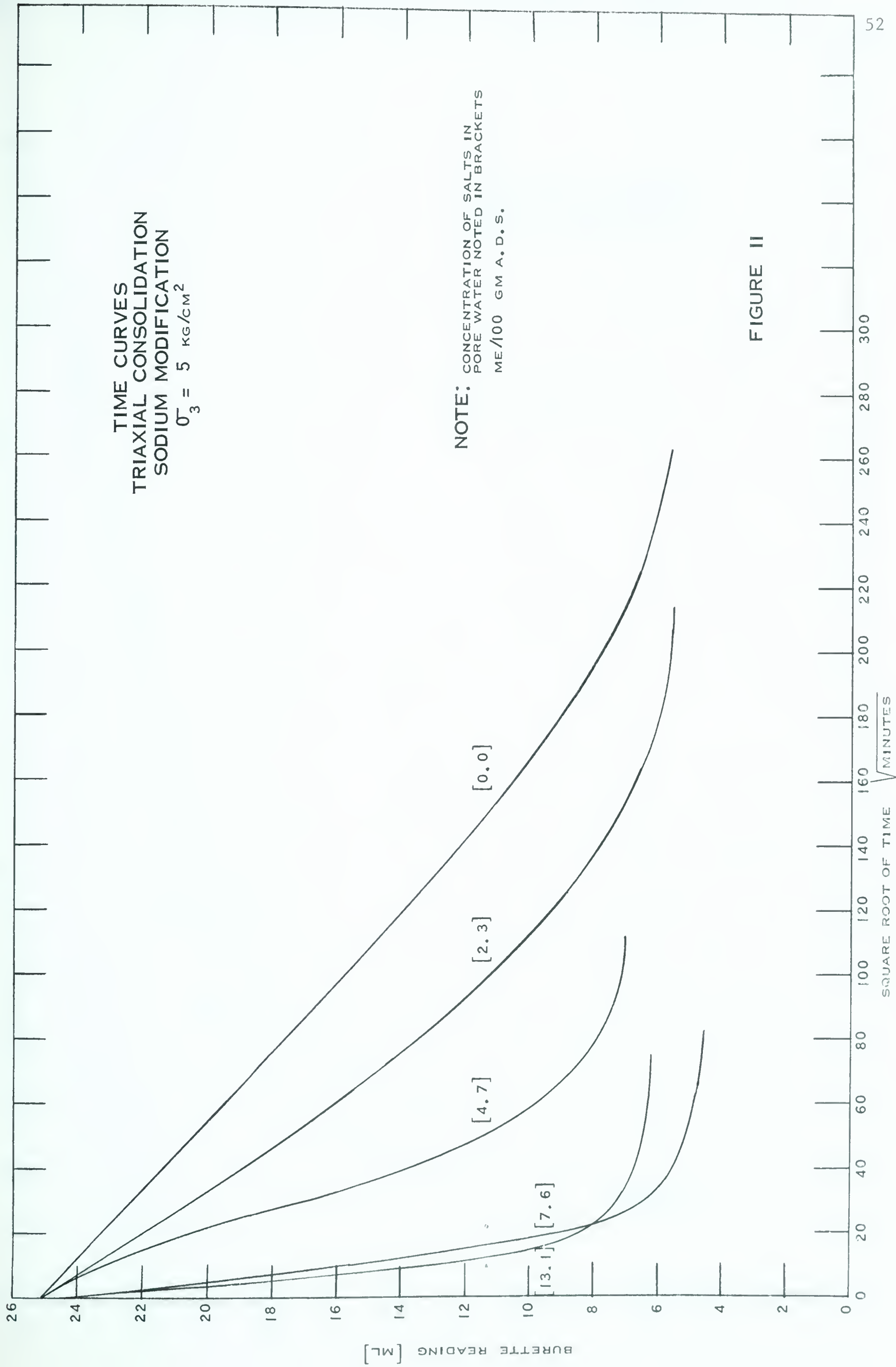


FIGURE II

CONSOLIDATION TIMES
VS SALT CONTENT
SODIUM MODIFICATION

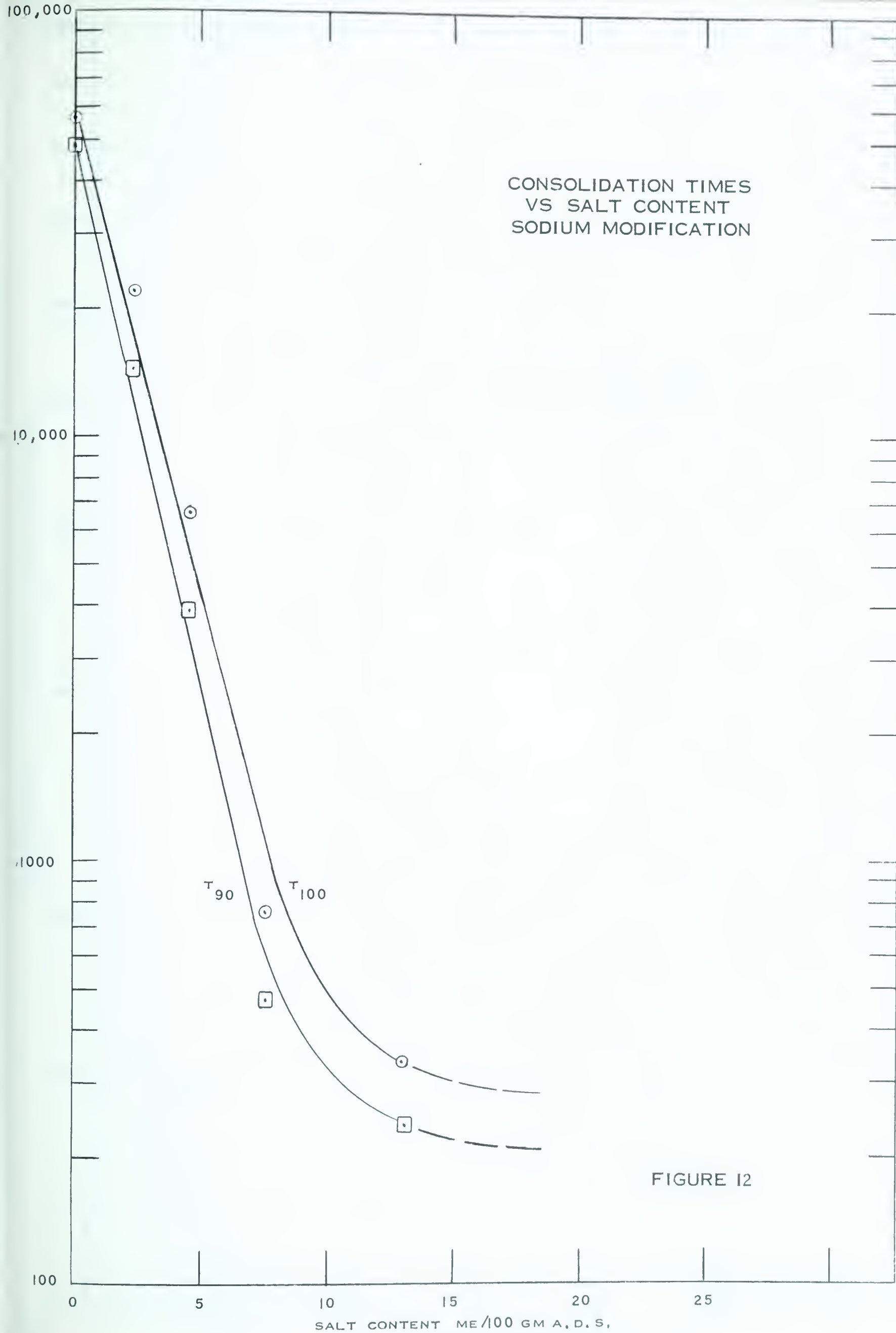


FIGURE 12

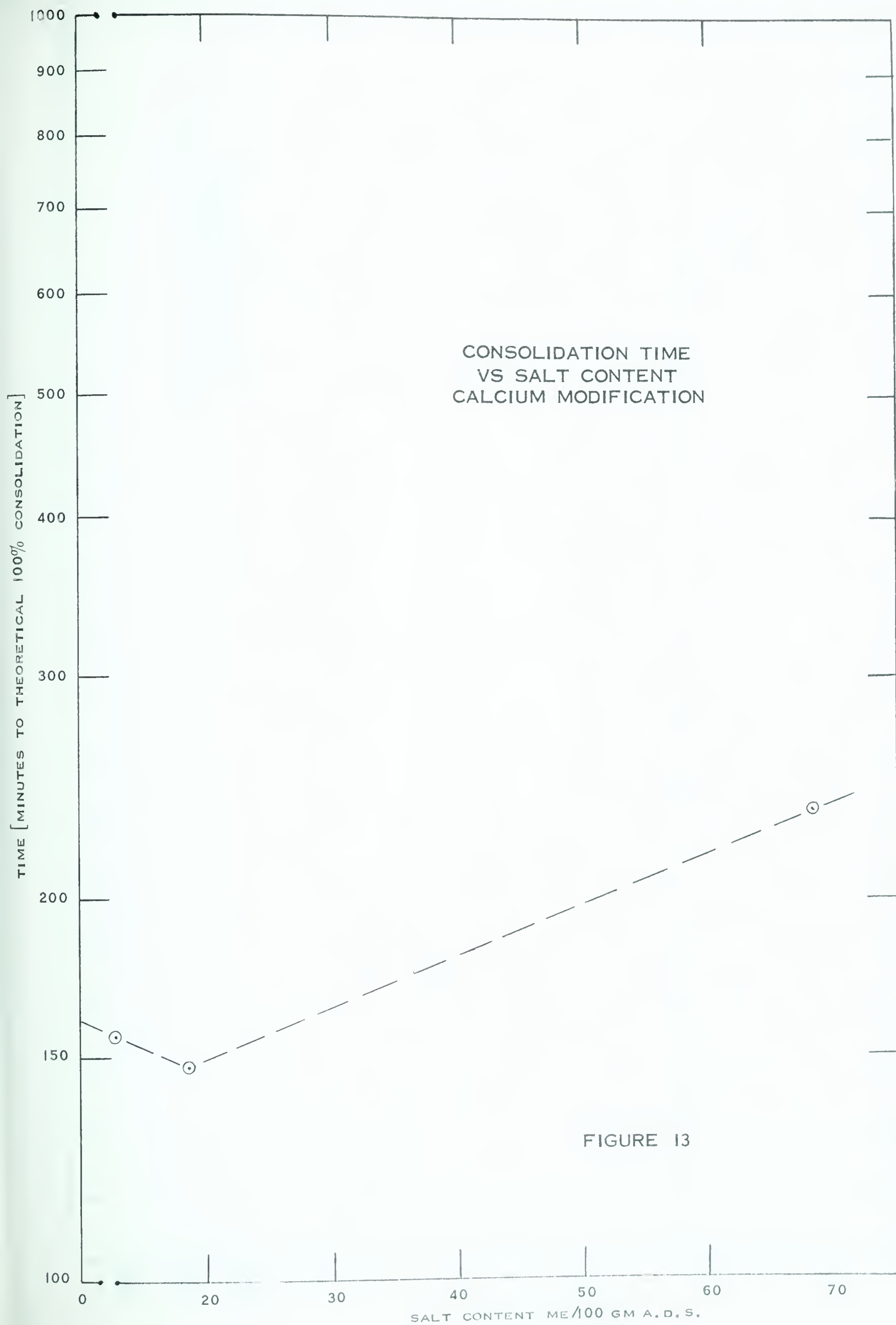


FIGURE 13

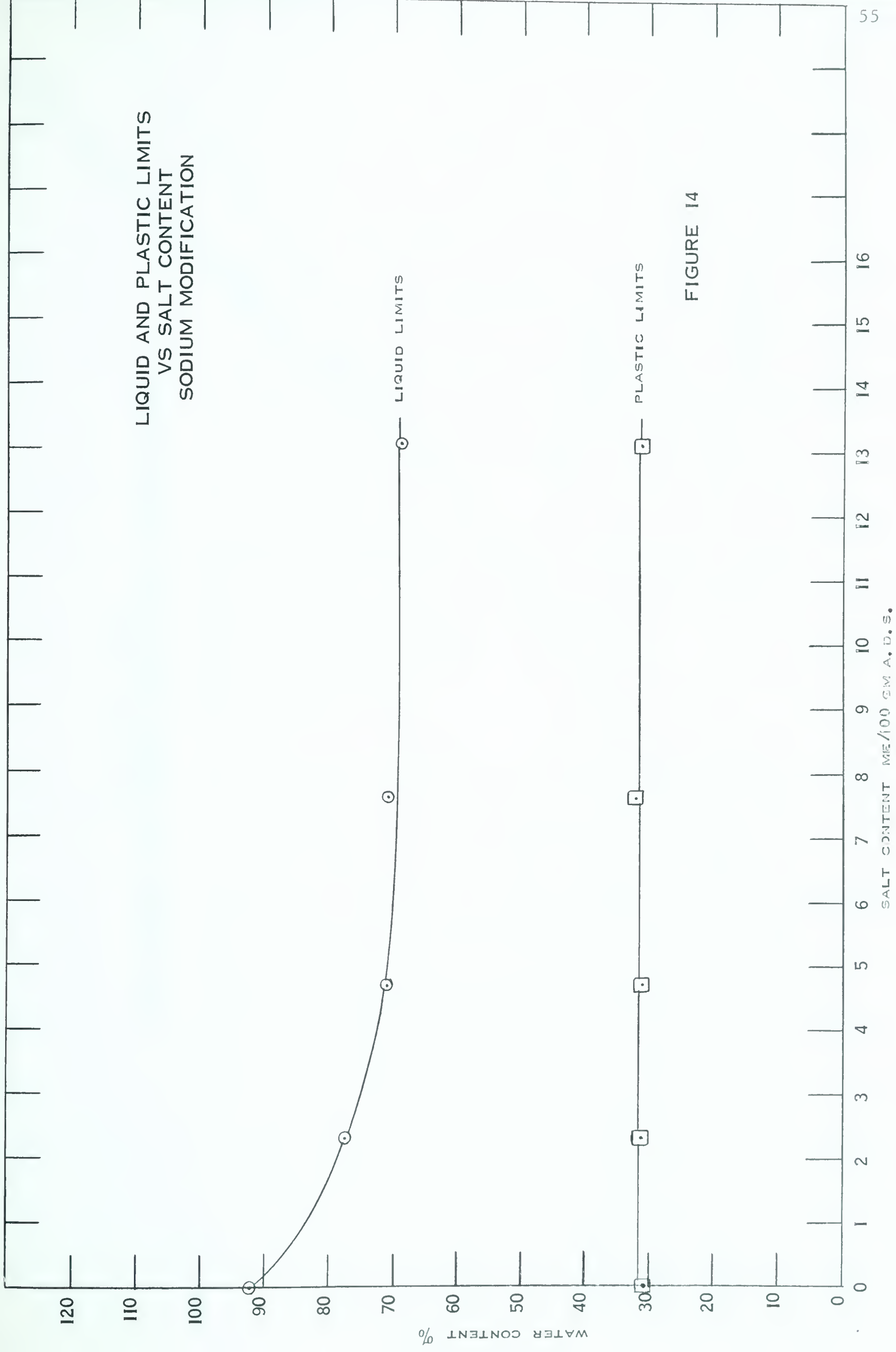
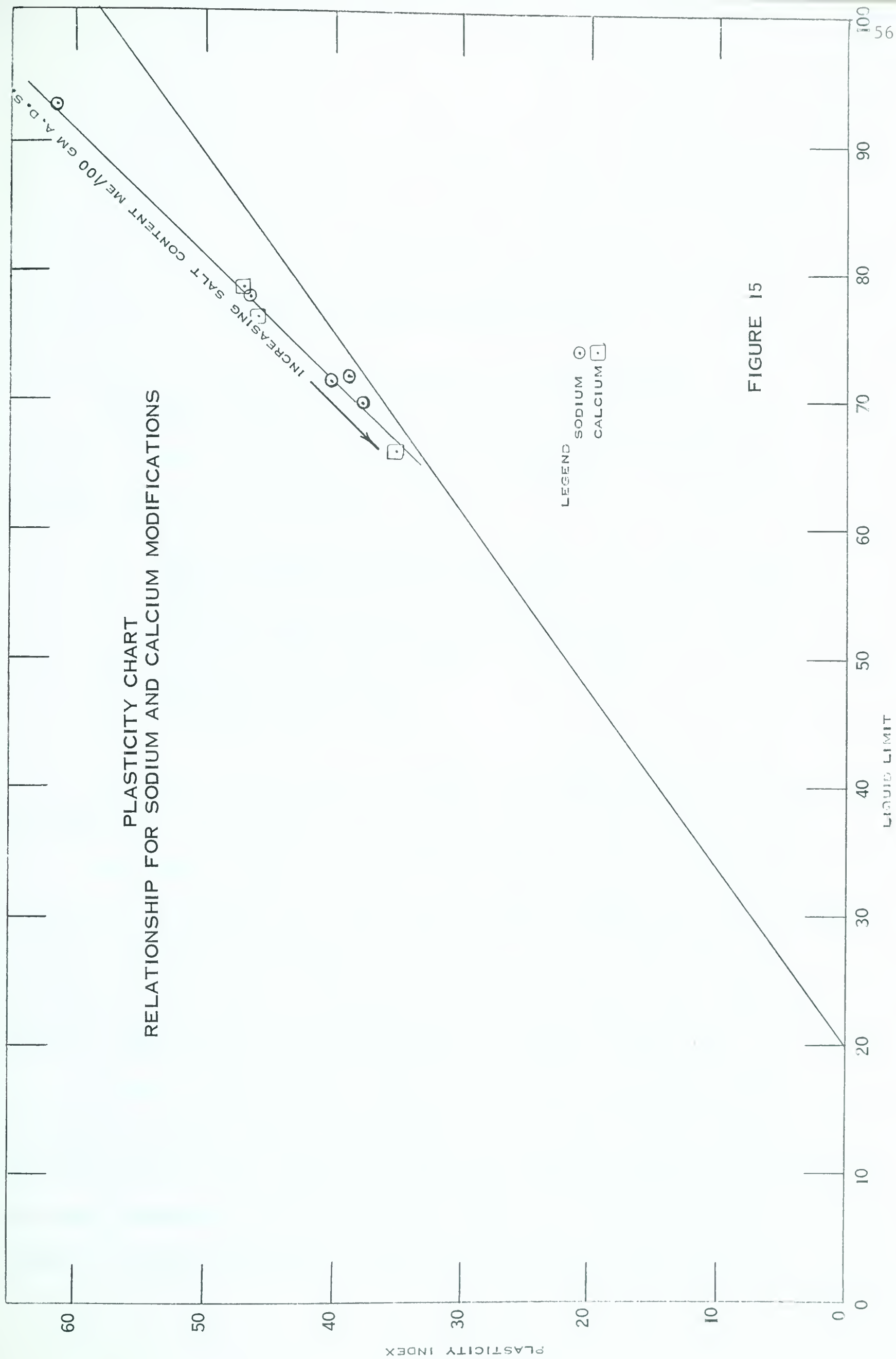


FIGURE 14



CHAPTER VI

DISCUSSION OF SHEAR STRENGTH RESULTS

6:1 Effects of Physico-Chemical Properties on Shear Strength

The factors which Lambe (1958, p.1654-30) considers to decrease the shear strength of clay are presented in Chapter II, but are repeated here for convenience in discussion. They are:

1. Reduction of electrolyte concentration
2. Cation exchange from high to low valence (e.g. Ca^{++} to Na^+)
3. Adsorption of anions (e.g. phosphate)
4. Exchange from a cation of small hydrated radius to a cation of large hydrated radius (e.g. Na^+ to Li^+)
5. Increase of dielectric constant of pore fluid
6. Increase of pH of pore fluid
7. Decrease of temperature
8. Increase in water content

The topic of this thesis is related to the first listed factor and the remainder of the variables may be accounted for in the following manner.

The effect of the adsorbed cation was kept constant by dealing with essentially an homionic soil. The change in shear strength with the adsorbed cation can be noted by comparing the shear strengths for sodium-soil and calcium-soil with equal concentration of salts in the pore water. The only directly comparable data for this situation is given in Table VIII.

It can be seen that the values of ϕ' and ϕ are essentially equal for each soil modification of this specific salt content. This is due to the fact that this salt content is close to providing the maximum values of the angle of shearing resistance for both soils. At lower salt contents there is a marked difference with the sodium soil having the lower shear

strength (Thomson, 1963; Taylor, 1959, p.27).

The effect of the adsorption of anions was eliminated by using the acetate radical. As the acetate salt was used in all cases, it has been assumed that any variations in the shear strength due to the anions was eliminated or reduced to a negligible amount.

TABLE VIII
ANGLES OF SHEARING RESISTANCE FOR SODIUM AND CALCIUM SOILS
AT SAME SALT CONTENT IN PORE WATER

Soil	Conc. of Salts in Pore Water, me/100 gms/a.d.s.	ϕ'	ϕ
Sodium	13.1	23°57'	13°10'
Calcium	13.0	23°08'	13°15'

Variations in shear strength caused by exchange from a cation of small hydrated radius to a cation of large hydrated radius, both having the same valence, are non-existent in this programme.

Lambe (1959, p.61) doubts that the increase of dielectric constant of pore fluid will prove to be of much practical importance to the soils engineer. The dielectric constant will be constant at a value of about 80 as the same distilled water was used throughout (Timm, 1950, p.344).

Clays suspended in solutions of pH above 7-8 can cause an increase in the cation exchange capacity due to dissociation of $\equiv\text{SiOH}$ groups present at the edges of the clay mineral. At pH values below 7 a decrease in exchange capacity of the clay can occur due to the adsorption of protons, (Taylor, 1959, p.21). Spot checks on the pH values of expelled water

from consolidation processes in this research found the pH to vary from 7.1 to 8.3 for the sodium and calcium soils. It seems reasonable to assume that this variation would not influence the exchange capacity sufficiently to produce a significant variation in the shear strength. Taylor (1959, p.21) states that the effect of pH variations is much more marked in kaolinitic clays than in illites or montmorillonites, as a small change in the exchange capacity of an illite or montmorillonite clay is a very small fraction of the whole. The soil used in this thesis is mainly montmorillonite and illite, therefore the pH likely has a minor influence on the total exchange capacity.

Temperature has been considered to be a variable influencing shear strength but in recent research (Semchuk, 1962, p.119) temperature was found not to be a significant factor for the soil used in this programme in its natural remoulded state.

The moisture content does not change once the triaxial test has commenced as constant volume is maintained. The moisture content at failure is, of course, related to the compressive strength and this is taken into consideration.

In the light of Lambe's views therefore, it appears reasonable to say that the only variable that can significantly affect the shear strength in this thesis is the electrolyte concentration. The influence of this variable follows from the Gouy-Chapman theory of the diffuse double layers which Lambe (1958, No. 1654) considers to affect shear strength.

6:2 The Clay Mineral, Diffuse Double Layer and Adsorbed Water

In studying soils one generally considers a three phase system consisting of soil particles, air and water. In this research programme

essentially saturated samples are used thus the air phase may be neglected. Due to the electrical unbalance of the crystalline clay mineral, a soil water system is established which consists of a central solid phase (the clay mineral), a surrounding double layer (the ion swarm), oriented water included in the double layer, and an outer liquid phase (electrolyte solution; pore water) (Pawluk, 1962, p.5)¹.

The charge deficiency of the clay mineral arises from three fundamental structural characteristics: (1) isomorphous substitution, (2) broken bonds on crystal edges, (3) exposed hydroxyl groups (Grim, 1953, p.132-134).

The nature of the double layer depends upon the surface charge density* of the solid phase, its bonding energies*, kind of adsorbed ions and valences, and concentration of electrolytes in solution. The most accepted theory regarding the double layer and the forces involved with it is that presented by Stern. This concept is a unification of the Gouy-Chapman theory and Helmholtz's postulates. The ion and potential distribution in the double layer in accordance with Stern are illustrated schematically in Figure 16. In order to establish electro-neutrality, cations are attracted to the clay particle to form the double layer system. The double layer system is comprised of an inner monolayer and an outer diffuse layer. The thin inner layer consists of the charge deficiency within the solid phase and the monolayer of adsorbed ions on its surface. This monolayer is tightly held and includes a thin layer of immovable liquid. The outer diffuse layer contains ions which are attracted to the clay particle by the charge deficiency but are prevented from swarming to

¹ Discussion of diffuse double layer after Pawluk (1962)

DIAGRAMMATIC REPRESENTATION OF THE ION AND POTENTIAL DISTRIBUTIONS IN THE DOUBLE LAYER ACCORDING TO STERN'S CONCEPTS

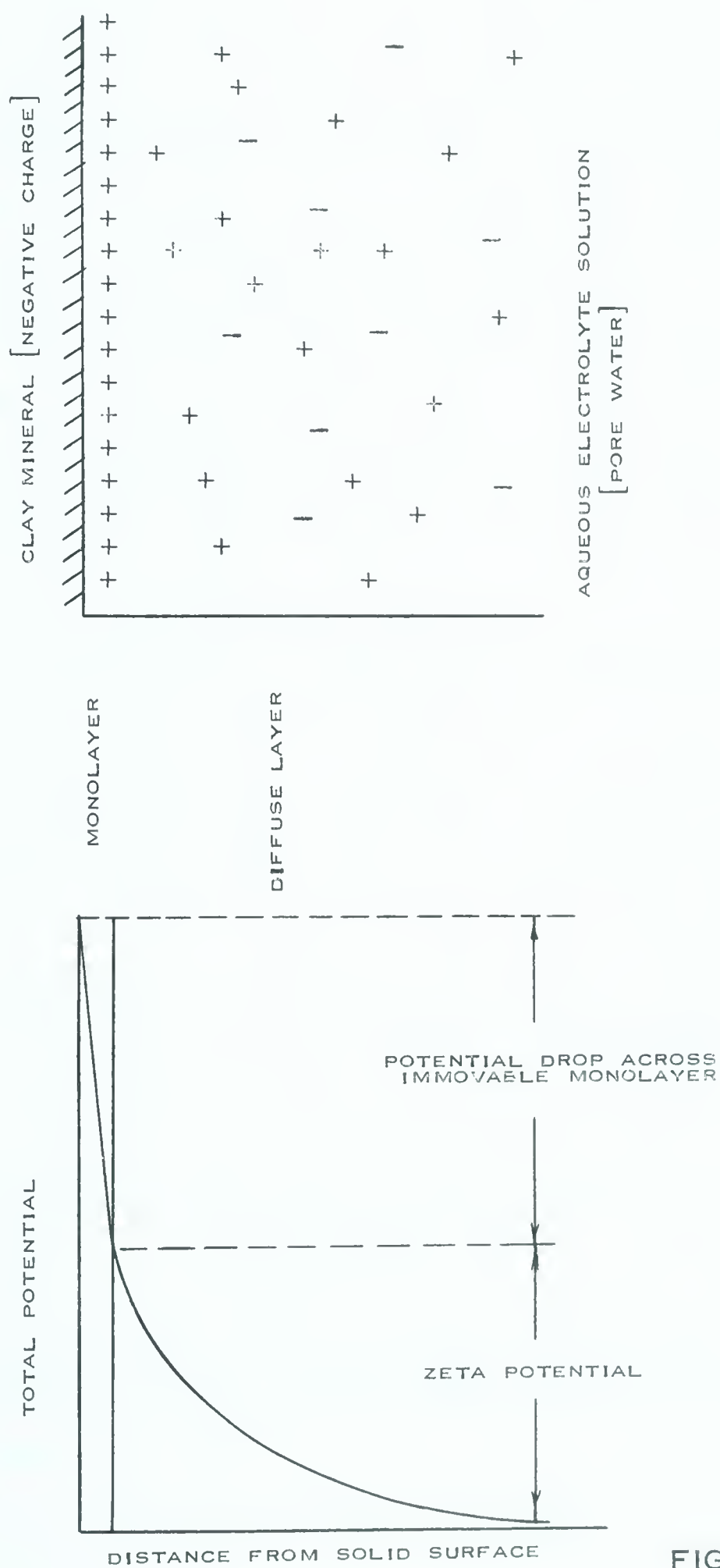


FIGURE 16

the particle by thermal energy and forces due to other ions and colloids. The concentration of ions in the double layer decreases with increasing distance in a direction perpendicular to the plane of the surface until it becomes equal to the electrolyte concentration of the pore water. A potential drop exists across the monolayer, and between the monolayer and the pore water there exists another potential drop. This latter potential drop is referred to as the zeta potential (Figure 16).

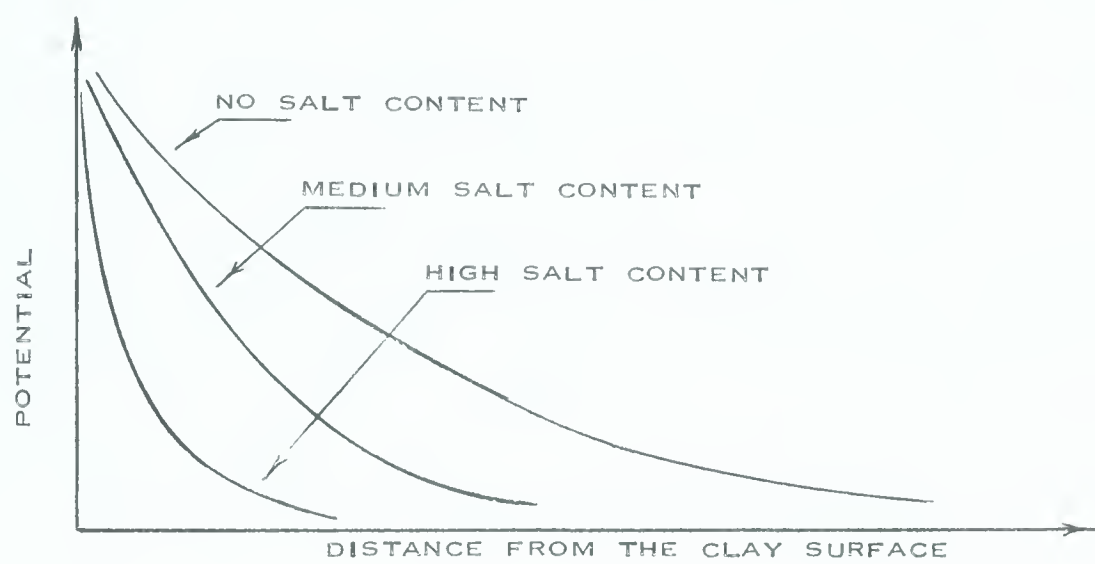
The clay mineral can possess adsorbed water on the basis of the dipolar nature of the water molecule. When a clay particle is exposed to water, the exchangeable cations hydrate first, if they are capable of hydrating (Low, 1961, p.322). Then the soil particle hydrates by the formation of hydrogen bonds between the water molecules and the clay mineral. The first layer of adsorbed water is held with its positive end turned in towards the negative clay mineral and having its negative end extending outward. Therefore, this first layer forms a new seat of negative charges upon which can be built another layer of oriented water molecules. This process of building up layers could be continued indefinitely if it were not for the facts that: (1) water molecules possess thermal energy and tend to be in a state of continuous motion thereby preventing regular orientation (Grim, 1953, p.163); (2) the competition between adsorbed ions and clay mineral surface atoms for the molecules is too great (Low, 1961, p.322). At the clay mineral surface, therefore, the water molecules will be highly oriented to form a "rigid" state, and the degree of orientation will decrease outward thereby reducing the rigidity of the adsorbed water structure.

The adsorbed ions meanwhile promote disorder in the water hull (Low 1961, p.323). If the ion is small and monovalent, e.g. Na^+ , it may move throughout the adsorbed water hull without causing disorder.¹ Due to this fact, the presence of Na^+ as the adsorbed ion allows the formation of very thick adsorbed water layers if the water is available. With Na^+ the oriented water net is not excessively rigid, but is very thick and there is a gradual transition from the oriented water to the free water (Grim, 1958, p.21). With a large or multivalent ion, eg. Ca^{++} , there is a large disordering effect because the large hydrated Ca^{++} cannot move throughout the oriented water with ease. Therefore, when Ca^{++} is the adsorbed cation, the water layer is thin and there is a sharp break between the oriented water and the pore water.

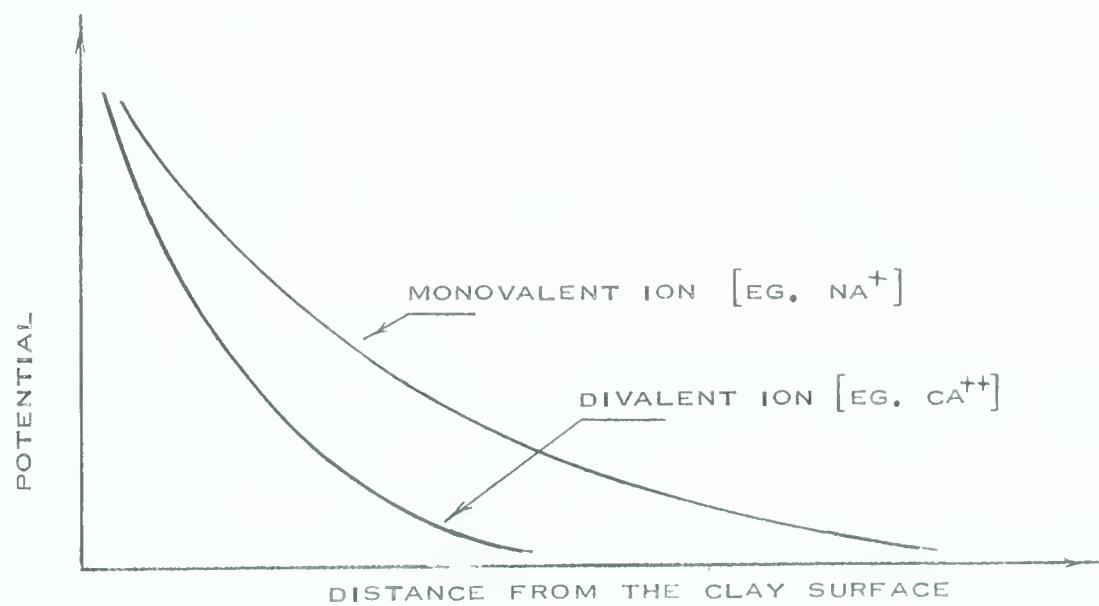
When the system is in dynamic equilibrium the same number of ions are migrating in and out of the double layer per unit of time. However, if the concentration or composition of electrolyte solution is altered an exchange of ions between the liquid phase and the double layer occurs, and a new equilibrium will be established. That is, if the concentration of cations in the pore water is decreased, an increasing number of cations will dissociate away from the clay mineral, the zeta potential will increase and the adsorbed water hull will increase in thickness. An addition of cations to the liquid phase will cause the cations in the diffuse layer to come closer to the clay mineral; the zeta potential will decrease and the water hull will decrease in thickness. Figure 17 shows representations of the double layer when subjected to different cations and salt concentrations.

¹. The Na^+ ion does not hydrate in the diffuse double layer (Grim and Cuthbert 1945, p.9).

INFLUENCE OF TWO VARIABLES ON THE DIFFUSE LAYER



INFLUENCE OF THE CONCENTRATION OF SALTS IN THE PORE WATER ON THE DIFFUSE LAYER



INFLUENCE OF THE VALENCE OF THE CATION ON THE DIFFUSE LAYER

FIGURE 17

The nature of the adsorbed water depends directly upon its structural development. There is some controversy regarding the structure of adsorbed water. Grim (1953, p.171) reports various values for the density of the adsorbed water and these range from 0.73 to 1.7.

Low (1961,p.323) states:

"The scientific volume, viscosity, and freezing resistance of the water are directly related to its structural development. Consequently, the magnitudes of these properties decrease continuously with distance from the mineral surface. The same applies to the relative thermodynamic properties. It should be noted, however, that nowhere is the water structure so rigid that ions cannot diffuse through it; nor is it so rigid that it will not shear under stress. Nevertheless, it has a yield point, which depends upon the proximity of adjacent mineral surfaces, and it exhibits non-Newtonian behavior at low hydraulic gradients. Although the precise molecular arrangements of water cannot be specified, it is not that in ice".

Therefore, one may say that the adsorbed water is not like the free water in its properties and is able to withstand some stress.

6:3 Shear Strength of Sodium Modification

The presence of sodium cations as the adsorbed complex results in the development of thick layers of oriented water with a gradual transition to non-oriented water (Grim, 1948, p.8). As salts are added to the pore water, a new equilibrium is established and the water hull decreases in thickness.

Both the total and the effective angles of shearing resistance tend to increase with an increased concentration of salts in the pore water, however, the most pronounced variation was with the effective angle of shearing resistance (ϕ'). The values of the angle of shearing resistance appear to vary in accordance with the thickness of the water hull, i.e., as the salt content increases, the adsorbed water hull decreases in thickness

and the shear strength increases. At the highest salt content studied, there was a slight drop in the angles of shearing resistance. This drop may or may not be due to experimental error. The author is uncertain as to whether the angles of shearing resistance will have a minor increase or decrease at higher salt contents, but in any case, the change is likely minor as compared to the strength increase at lower salt contents.

One might explain the variation in shear strength by following the trend of thought developed by Lambe(1960, p.561). His basic equation¹ relating shear strength ot interparticle forces is:

$$\sigma_n' = \sigma_T - (u + R - A)$$

The repulsive forces decrease with increasing distance at approximately an exponential rate while the attractive forces decrease in magnitude as the square of the distance (Lambe, 1953, p.20). Therefore, at very small interparticle distances "A" will be larger than "R". With zero salts in the pore water, the adsorbed water hull will be thick and the repulsive forces will be larger than the attractive forces. This situation will result in a low shear strength. An increase in the electrolyte concentration will depress the double layer resulting in higher shear strength due to the increased attractive forces and decreased repulsive forces.

Figure 2 illustrates the fact that the angles of shear resistance do not vary in linear proportion to an increase in salt content. This is likely due to the fact that at low salt contents the repulsive forces dominate, then as the salt content increases the attractive forces increase

¹. equation and symbols previously discussed in Chapter II

slowly until the interparticle distance becomes relatively small and they are able to overcome the repulsive forces rapidly. The relatively rapid increase of shear strength at 7.6 me/100 gms. a.d.s. might be explained in this manner.

Many workers correlate a type of structure with the forces which are prevalent (Lambe 1958). A reduction in "R" accompanied by an increase in "A" will result in a flocculated structure and large "R" values and small "A" values provide a dispersed structure. A dispersed structure may have occurred in the samples which are remoulded at low salt content. With a high salt content the particles would tend to form a flocculated or "card-house" structure. Changes in particle orientation or structure have large effects on shear strength (Scott, 1963, p.340). The structure will also be influenced by loading, e.g..moulding pressures, triaxial loads. Shear strength is generally higher for a flocculated structure than for a dispersed structure. Therefore, if a dispersed structure occurred with a low salt content, a low shear strength would be expected. On the other hand a higher shear strength would occur with the flocculated structure developed from a high salt content. However, in order to validate an explanation of shear strength on the basis of structure, the author feels that fabric studies should be undertaken (Quigley, 1962), therefore only brief mention of structure is made here.

Pore pressures did not develop to as high a value with respect to the deviator stress for the low salt contents as for the high salt contents (indicated by the low \bar{A} value). Referring to Table IV one can see that the average \bar{A} value was higher for the higher salt contents. The reduced value of \bar{A} indicates that there is less tendency for the soil skeleton to

tend to reduce in volume due to shear strains at low electrolyte concentrations. In order to account for the lack of a tendency for the soil skeleton to tend to decrease in volume one may imagine the formation of a "pseudo-structure". With the presence of large zeta potentials, it seems reasonable to assume that the double layers and adsorbed water hulls will overlap. The most important result of interaction between double layers is the decrease effected in the charge of the double layer which results in an increase of free energy of the double layer (Lambe, 1953, p.17). Since systems tend to exist in a state of minimum free energy, the colloids repulse each other when their double layers interact. The magnitude of the double layer and the zeta potential determine the thickness of the adsorbed water hull. Therefore, in the case of relatively thick water hulls, it appears conceivable to state that the water hulls will overlap so as to form a "pseudo-structure". The shear strength will remain low due to the strong repulsive forces.

An applied load will be taken up by this "pseudo-structure" and very little stress will be thrown into the water phase. As the double layer decreases with higher concentration of salt, the water hull will become more strongly oriented and overlap will not occur to the same degree. The "pseudo-structure" will decrease in significance and applied loads will have their stress thrown into the water phase to build up pore pressure.

The differences in the relationships between deviator stress and strain (Figure 4), may be explained by this trend of thought. For low salt contents the deviator stress reaches a maximum at a low strain and failure occurs abruptly with a definite failure plane. If the previously mentioned "pseduo-structure" exists, its bonding forces are limited in

strength by the degree of orientation which can exist at large distances from the soil particle. Therefore, when the applied load reaches a certain magnitude, the "pseudo-structure" can collapse and cause failure to occur rapidly in a brittle manner. As salt contents increase and the "pseudo-structure" decreases in significance, failures tend to be of a plastic nature. This line of reasoning is substantiated by the fact that at low salt contents, definite shear planes were observed in the failed triaxial specimens, but plastic failure occurred at high salt contents.

Scott's presentations (1963, p.340) on a dispersed structure may also aid to explain the brittle failure. A dispersed structure has the particles existing in randomly oriented packets in which they are oriented parallel to one another. Shearing distortion causes rotation of the packets in the zone of shearing until all particles are parallel to the shearing direction. In the case of low salt contents, a dispersed structure and packet arrangement may exist. As the load is applied the packets will tend to try to become parallel and as soon as this condition is reached, failure will occur along the developed shear plane (brittle type failure).

6:4 Moisture Content, Void Ratio, and Shear Strength of Sodium Modifications

The data presented on Figure 3, Logarithm of Compressive Strength versus Moisture Content, illustrates distinctly that a linear relationship between these two factors exists for each salt content. As the concentration of salts in the pore water increases, the slope of this linear relationship decreases, and the curve tends to shift downward.

Since the triaxial samples may essentially be considered saturated, the void ratio may represent the abscissa of the plot instead of moisture content. In the region of the higher void ratios, the compressive strength shows an increase with a decrease in salt content. In the lower ranges of the graph where the void ratios are smaller, the curves cross one another so that no definite statement can be made regarding the compressive strength variation with respect to salt content. The increase in compressive strength with decrease in salt content appears to be an anomaly with respect to the variations in shear strength represented by the Mohr envelopes. Warkentin (1962, p.3) reports a similar situation when comparing shear strengths at equal void ratios. In studying Figure 3, it is also apparent that for one compressive strength the modified soil may have a variety of moisture contents depending upon the salt content. With higher void ratios, where the original confining pressure was low, the moisture content tends to decrease with an increase in salt content for one particular compressive strength. At lower void ratios, which result from higher confining pressures, the variation in moisture content for a particular compressive strength does not vary in any consistent manner. In fact, the points have a tendency to be somewhat grouped. The data in this region is definitely quite clustered when compared to that at lower confining pressures and higher void ratios.

In analyzing this data, it was found advantageous to present two other graphs, Figure 18, Final Void Ratios vs. Salt Content, and Figure 19, Triaxial Confining Pressure vs. Final Void Ratio. Figure 18 shows that the void ratios for each confining pressure decrease substantially with an increase in salt content. As the salt contents increase, the void

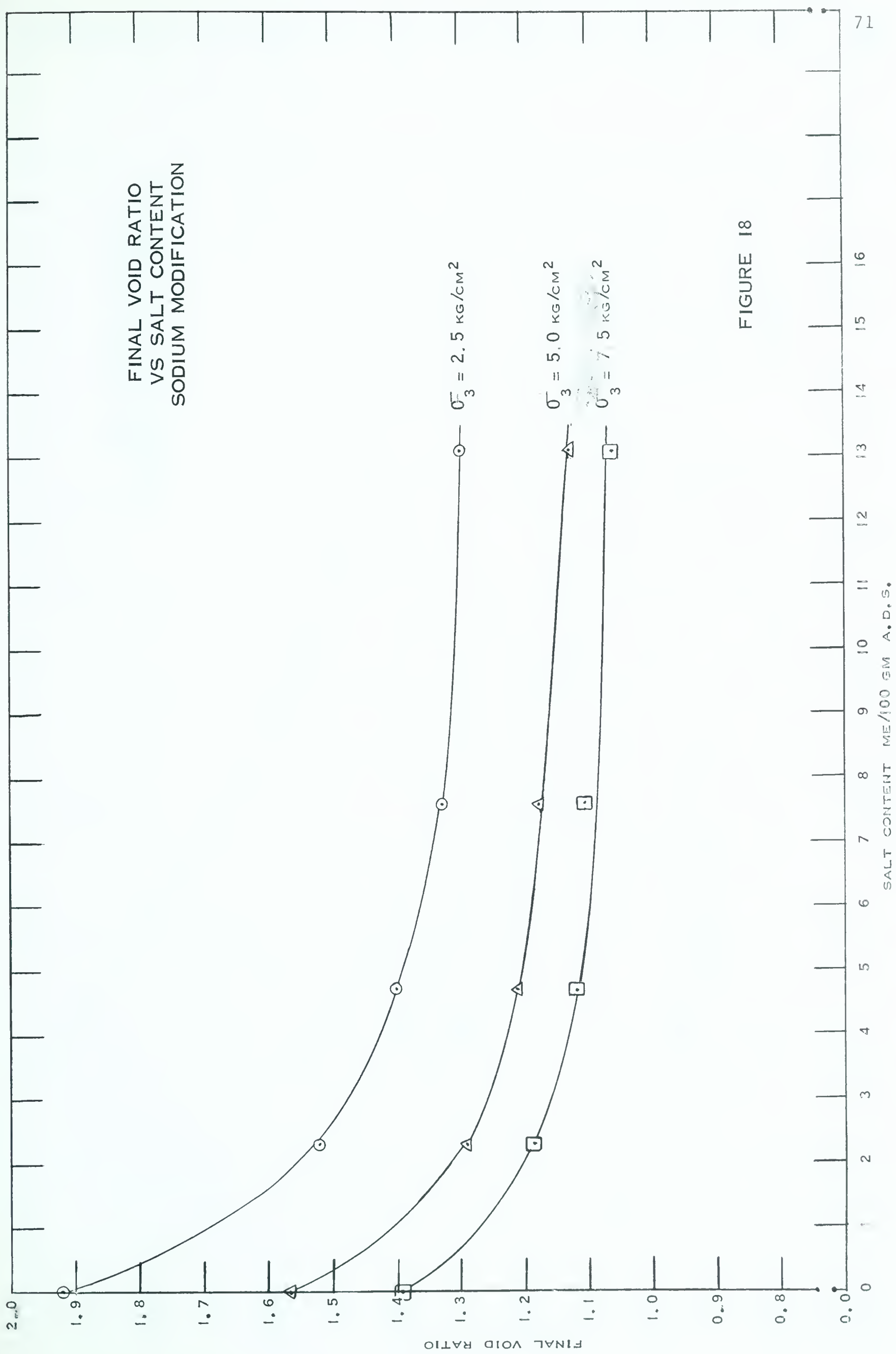


FIGURE 18

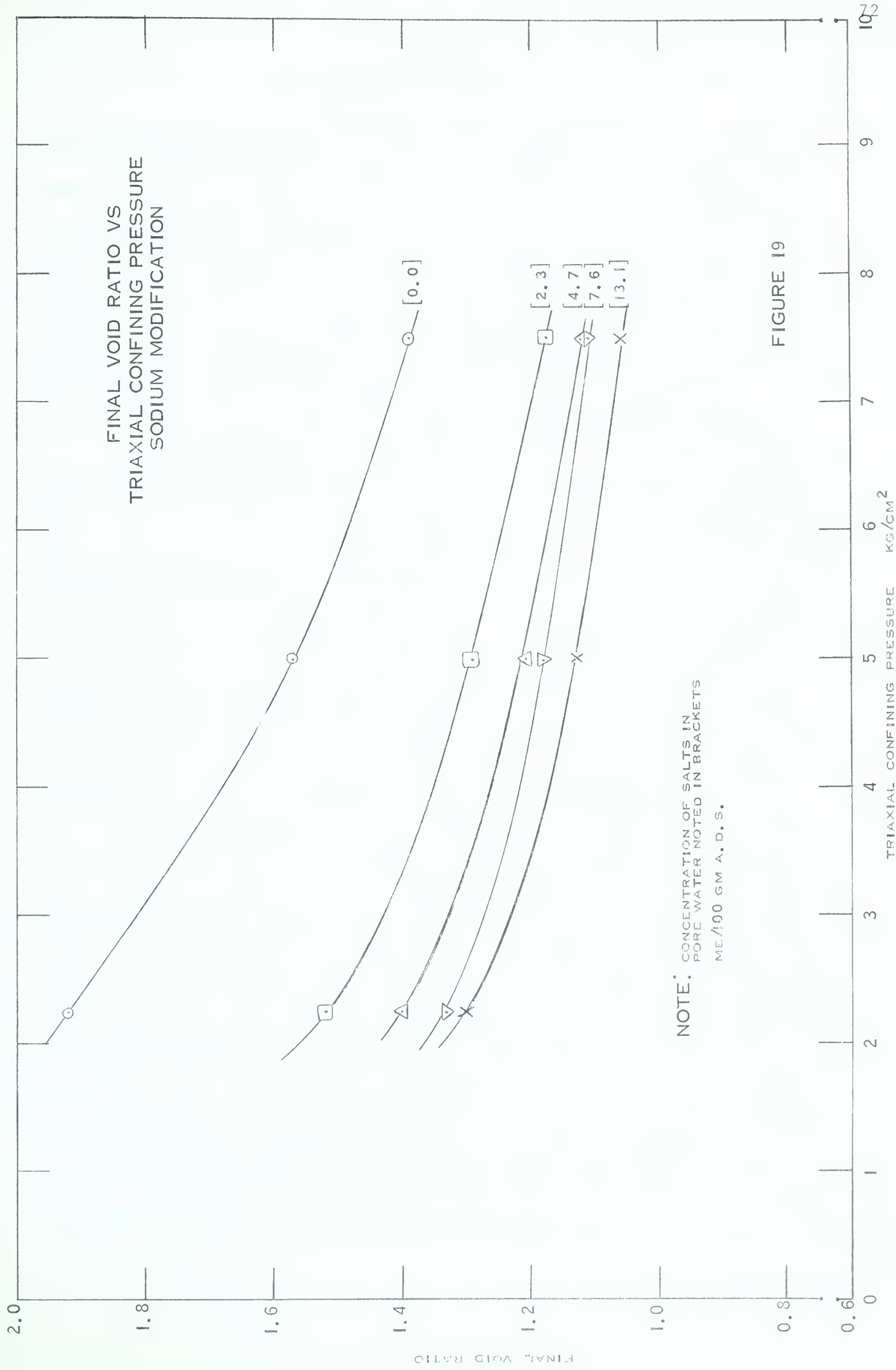


FIGURE 19

ratios approach each other in value and at high confining pressures they become practically identical. The lower the salt concentrations and confining pressures, the higher are the void ratios. This variation in void ratio is synonymous with the change in moisture content previously discussed. In Figure 19 the data presented are the same as in Figure 18, but this plot is presented as it facilitates an understanding of the trends by illustrating clearly the role of the confining pressure.

The displacements of the curves in Figure 3 ($\log \sigma_1 - \sigma_3$ vs m.c.) appears to correlate with the decreasing thickness of the adsorbed water hull. At low salt contents the water hull is thick resulting in high void ratios and high moisture contents. A decrease in the moisture content and the void ratio is brought about by two factors: (1) an increase in the concentration of salt in the pore water which tends to decrease the zeta potential and the thickness of the water hull; (2) an increase in the confining pressure which is an external force able to cause the expulsion of additional water.

The sodium modification with zero salts in the pore water has a thick water hull (Grim, 1958, p.20) which requires a very high confining pressure to acquire a void ratio comparable to that occurring when the water hull is thinner due to electrolyte concentration. Therefore, when it is stated that the compressive strength is higher for zero salt content for equal void ratios, it must be kept in mind that in order to obtain this void ratio, very high confining pressures are required. In studying Figure 19, Triaxial Confining Pressure vs Void Ratio, it becomes apparent that for the modification with zero salts to reach the void ratios of approximately 1.15, an extremely high confining pressure would be required

(approximately 15 Kg/cm²).

The shear strength seems to be influenced by physico-chemical phenomena at low salt contents and low confining pressures to a much greater degree than at high confining pressures and high salt contents. When the soil is subjected to the former conditions, small changes in the salt content or confining pressure cause disproportionately large changes in strength properties. This fact is likely due to the repulsive and attractive force complex (Lambe, 1960) which seems to account for the shear strength. At high salt contents and confining pressures the water hulls have decreased in thickness and the shear strength values appear to group as illustrated in Figure 3. In this region the void ratios for the salt contents are nearly equal. Small changes in the salt content do not appreciably change the void ratio in the presence of high confining pressures. It may be surmised that the attractive and repulsive forces do not predominate in the variation of the shearing strength in the latter conditions. These forces are still present but it seems possible that when the void ratio decreases to a certain region that other factors exert a relatively greater influence on the shear strength, e.g. inter-particle friction, particle shape, etc.

These postulates, indicate that physico-chemical phenomena are more important in the region of high void ratios and low confining pressures. Here salt content and confining pressure must be considered when relating shear strengths. At low void ratios other factors exert their influence on shear strength and the physico-chemical phenomena exert a relatively small influence on shear strength characteristics.

To illustrate that these low confining pressures are still of practical significance, one may consider a confining pressure of 2 Kg/cm^2 and a soil mass having a total unit weight of 100 lbs. per cu. ft. Under these assumed conditions the depth that these physico-chemical phenomena are of interest is at least in the order of 40 ft.

6:5 Shear Strength for Calcium Modifications

For the calcium modifications, the variation in shear strength can be considered to be within the range of experimental error for the salt contents studied, i.e., 13.0, 18.9 and 68.3 me/100 gms. a.d.s. Unfortunately due to insufficient washing of the calcium-clay, the condition of zero salts in the pore water was not achieved. As the determinations of salt contents were in the last stage of the testing programme, there was insufficient time to do further washings and perform additional triaxial tests. Therefore, only shear strength values are known for the high concentrations of salts. The angles of internal friction for the natural washed clay were lower than the calcium due to the fact that there was less calcium adsorbed on the natural material. The presence of extraneous adsorbed cations on the natural soil may have influenced the angles also.

When calcium is the exchangeable ion the oriented water hull is relatively thick and the transition to non-oriented water is abrupt (Grim, 1948, p.9). Due to the thin hull, the development of a "pseudo-structure" as previously envisaged is unlikely. Also it is reasonable to assume that due to the already thin water hulls it would be extremely difficult to further suppress the adsorbed water hull by the addition of salts to the pore water; especially within the range of salt contents

studied. The lack of variation in the shear strength within the range of salt contents studied parallels the idea of a water hull with a relatively stable thickness.

The thin water hull allows relatively high attractive forces and low repulsive forces thereby permitting comparatively high angles of shearing resistance to develop. In terms of structure, it might be stated that the calcium-soil has a flocculant structure due to the high attractive force. A flocculant structure is normally associated with higher shear strengths. Failure developed in a plastic manner similar to the high strength sodium samples.

The variation in salt content did not influence the development of pore pressure or the stress-strain characteristics. The pore pressure parameter \bar{A} has an average value of 88% indicating that there is a tendency for the soil skeleton to reduce in order to establish pore pressure.

The plot of the logarithm of compressive strength versus moisture content (Figure 9) shows little variation in the slopes of the linear relationships for the different salt contents. There is, however, a downward shift of the curves as the salt concentration increases. This tendency indicates that as the salt content increases, a comparable compressive strength is obtained at decreasing moisture contents. A plot of final void ratio versus salt content (Figure 20) shows that at low confining pressures the final void ratio tends to decrease with increasing salt content. Suspicion concerning shear strength variations might arise from these considerations, i.e., at lower moisture contents the shear strength should be higher; a fact which the presented Mohr envelopes dispute.

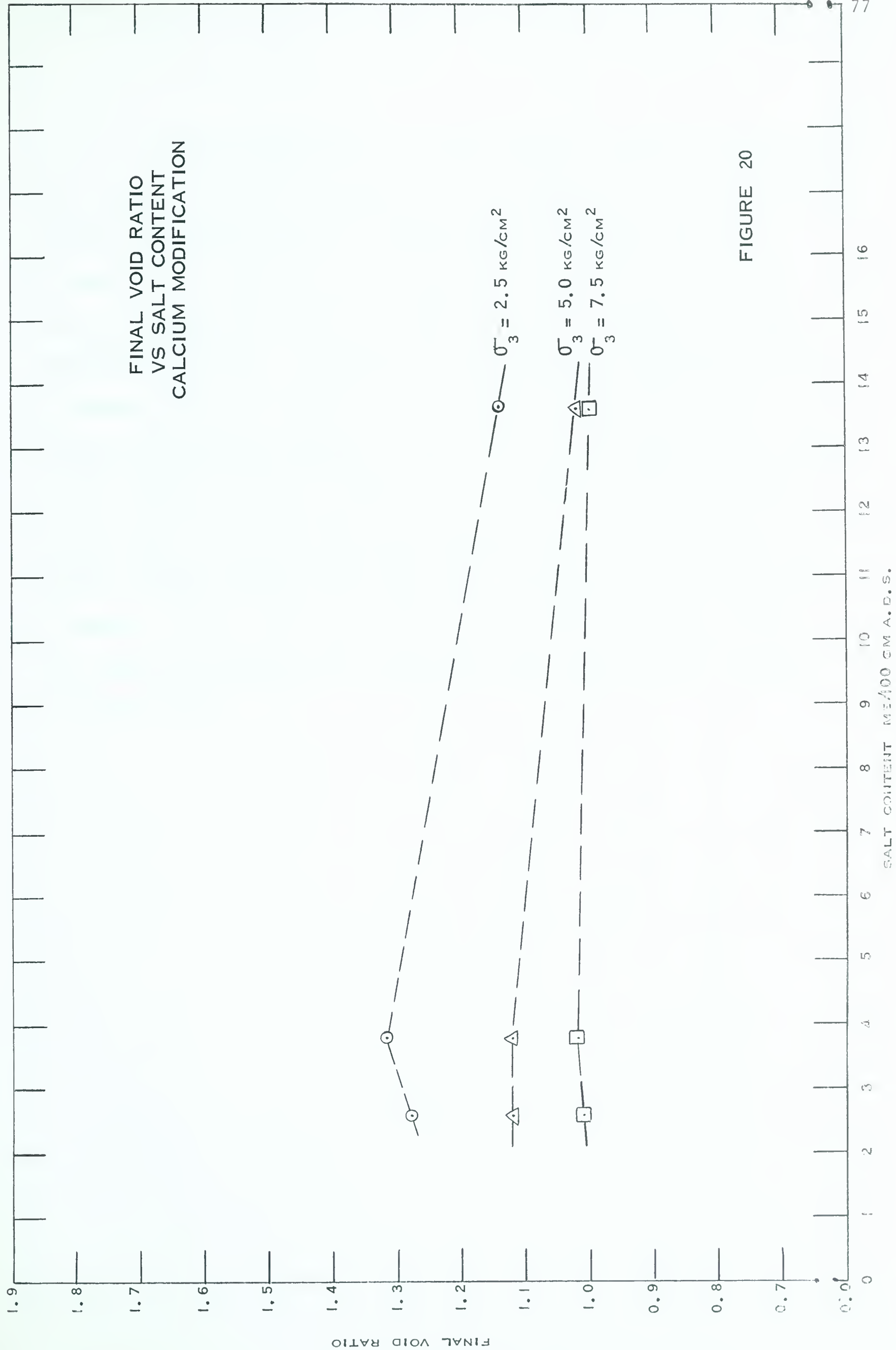


FIGURE 20

It may be argued that at the salt contents studied, the attractive and repulsive forces are combined so as to provide the maximum value of the angle of shearing resistance. The decrease in moisture content is insufficient to either alter this situation or allow any other forces, e.g., interparticle friction to increase in order to change the angle of shearing resistance. It appears that physico-chemical phenomena are more relevant at high void ratios. Figure 21 illustrates the decrease in void ratio with confining pressure for each salt content. When the salt content or the confining pressure reaches such a magnitude that the void ratios become small, the physico-chemical phenomena appear to become relatively small in their influence and cause little further variations in the shear strength. On the other hand, when the void ratios are large, as with the sodium modifications, the physico-chemical effect on shear strength is quite marked.

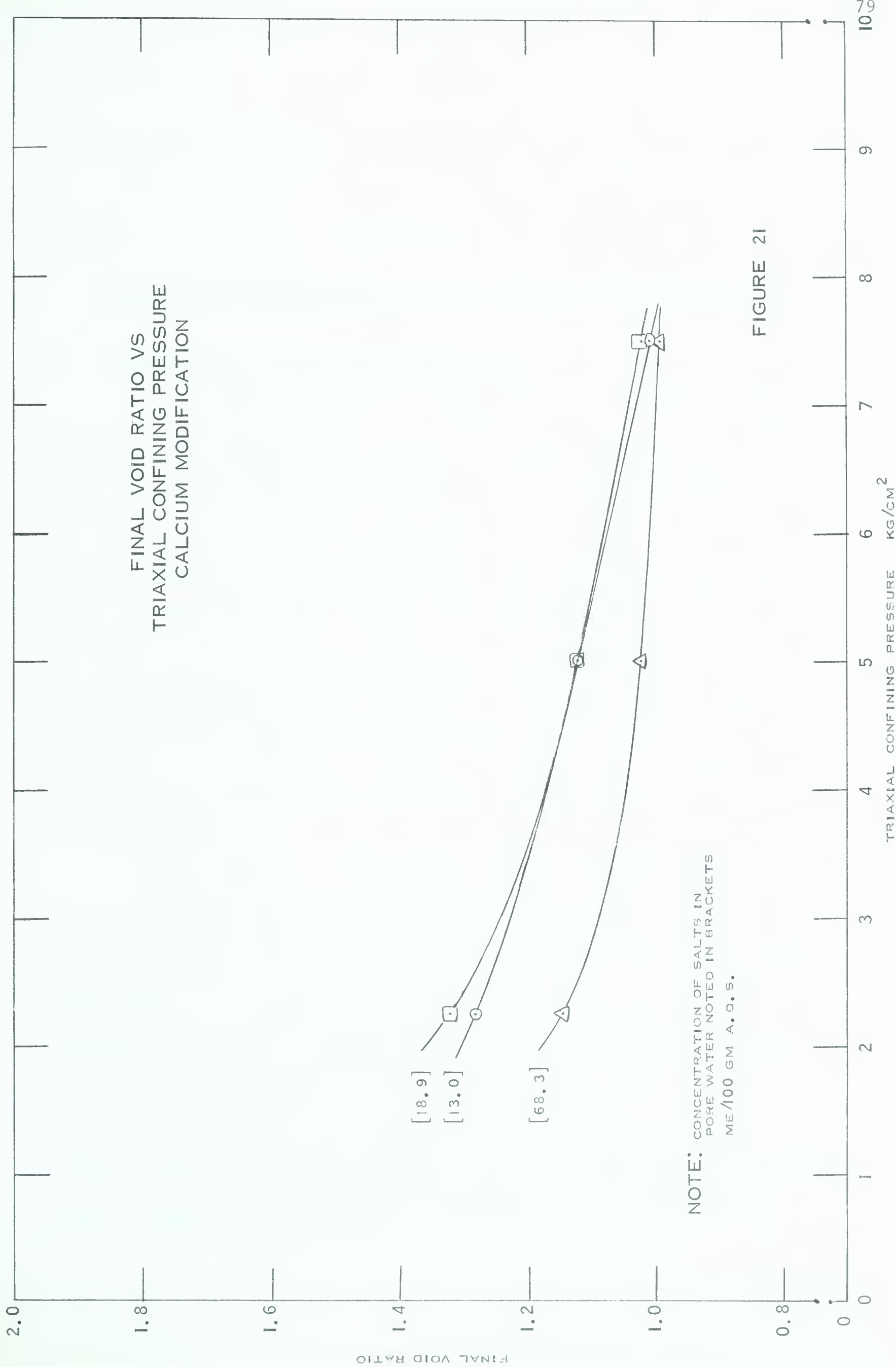


FIGURE 21

CHAPTER VII

DISCUSSION OF CONSOLIDATION, SPECIFIC GRAVITY & ATTERBERG LIMITS

7:1 Consolidation of the Sodium Modification

In an attempt to explain the large variations in time to reach theoretical 100% consolidation (t_{100}), the concept of the adsorbed water hull is considered (Grim, 1962, p.253). It has been stated that a low salt content results in a large water hull, which is associated with a high zeta potential. As the salt content increases, the zeta potential and the adsorbed water hull decrease. Specimens having zero salts in the pore water likely have a large percentage of their total water (adsorbed water and free water) present in the form of oriented adsorbed water. A pseudo-structure (Sec. 6:3) may have developed during the sample moulding process in which the samples were consolidated from approximately the liquid limit.

In the process of consolidation, water is expelled from the sample in order to reduce the excess hydrostatic pressure set up due to the confining pressure. Grim (1962, p.259) states "It can be considered that a small amount of pressure is adequate to remove the pore water and that considerable pressure is probably necessary to remove the oriented water". He also claims that an envelope of oriented water around the clay particles would make it relatively more difficult to adjust them in space one to the other with the application of pressure, than if the water was in the liquid form and could act as a lubricant (Grim, 1962, p.259). When a load is

applied to cause consolidation, the water is difficult to expell due to the formation of the "pseudo-structure". With time, however, the less oriented water leaves the sample, and the hydrostatic pressure is relieved slowly. The difficulty the soil particles have to adjust themselves also tends to increase t_{100} .

There is also a decrease in permeability with an increase in water hull thickness. Thick water layers reduce flow due to a reduction in pore diameter and due to the impending action of the bound water on the free water. (Grim, 1962, p.245). The pore diameter is considered to be that space in which free water might flow. The relationship governing the flow of water through round capillary tubes of small diameter is known as Poiseuille's Law. (Taylor, 1948, p.104). Poiseuille's Law is related to the pore diameter and the viscosity of the fluid. Tests performed by the Alberta Research Council showed that there is an increase in viscosity with an increase in salt content for the pore solutions. The author feels that the change in viscosity is not as significant as other factors. The pore diameter which varies with the thickness of the adsorbed water hull will influence the permeability as indicated by Poiseuille's Law.

In summary, consolidation time (t_{100}), is largely due to:(1) the decrease in pore size, (2) the character of the adsorbed water, (3) the resistance to reorientation of particles so as to provide better flow paths, (4) reduction in the number of flow paths.

Lambe (1958, p.1655-11) correlates compressibility to the charge on the particle, however, in the explanation of consolidation the nature of the fluid is likely as important as the charges present (Grim, 1962, p.261).

It seems feasible to explain the variation in t_{100} by the concept of adsorbed water as the process of consolidation is concerned with the expulsion of pore fluid.

The increase of salts in the pore water is associated with a decrease in time to reach theoretical 100% consolidation. The first addition of salts has a greater suppressive effect on the water film thickness than subsequent additions (Taylor, 1959). A plot of logarithm of time to theoretical 100% consolidation versus concentration of salts in the pore water is shown on Figure 12. The upper four points fall very close to a straight line. The plot curves in the lower extremities to fit the lower point (it is reasonable to assume that t_{100} cannot decrease to zero). The curve appears to become horizontal as Thomson (1963) reports $t_{100} = 180$ minutes for a salt content of 188 me/100 gms. a.d.s.

7:2 Consolidation of the Calcium Modification

The postulates which were presented for time to theoretical 100 percent consolidation for the sodium-clay do not appear to hold firm in the case of calcium. With the sodium-clay the values for t_{100} had a marked variation, however, the calcium-clay presents only a very slight variation in t_{100} for different salt contents. In the case of the sodium modification, t_{100} decreased with an increase in salt content, but with the calcium modification t_{100} values appeared to increase with increased salt content (confirmed by Thomson, 1963).

The plot of t_{100} versus salt content, Figure 13, may be open to interpretation due to the paucity of data. In comparing the curves presented for final void ratio versus salt content (Figure 20) with

Figure 13 it can be seen that the curves tend to be similar in shape but in an inverted position. The curves indicate that as void ratio increases t_{100} decreases and as void ratio decreases, t_{100} increases. Therefore, as t_{100} will depend on void ratio and permeability, the data supports itself.

7:3 Specific Gravity of Soil Modifications

The two factors which seem to influence the specific gravity tests to cause the marked variation in the results are the presence of salts in the pore water and the adsorbed cation complex. These factors will cause fluctuations in the determinations of specific gravity due to the adsorbed water hull and the presence of salts.

In order to discuss the variation in specific gravity, reference should be made to the equation concerned with the calculation of specific gravity in which a few modifications might be made.

(1) Original equation

$$G_s = \frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$$

where: W_s - denotes weight of soil

W_{b+w} - denotes weight of water and bottle

W_{b+w+s} - denotes weight of water and bottle and soil

(2) Modified equation

$$G = \frac{W_1}{W_1 + W_2 + W_3}$$

where: W_1 equals the sum of:

- (1) weight of soil particle devoid of all water except that in the crystalline structure.

(2) weight of adsorbed water remaining after over-drying
(105° - 110° F)

(3) weight of soluble salts present in pore water.

W_2 equals the sum of the weight of the bottle plus the water

W_3 equals the sum of:

(1) weight of bottle

(2) weight of soil particle devoid of all water except
that in the crystalline structure (submerged)

(3) weight of soluble salts present in pore water

(4) weight of free water in bottle (density γ_w)

(5) weight of adsorbed water (density γ_{aw})

One may now approach this equation from a purely hypothetical point of view. If a series of specific gravity tests could be performed with the only variable being that of the concentration of salts in the pore water, the only changes in the system would be the change in the thickness of the adsorbed layer and the weight of soluble salts. The density of the adsorbed water hull will be an influencing factor.

If the density of the adsorbed water is greater than that of the free water, then W_3 will increase. This will result in a decrease in the denominator of the equation and provide a greater value for G_s . Therefore, G_s will increase with an increase in the thickness of the water hull which is associated with a decrease in salt content.

If the density of the adsorbed water is equal to that of the free water, then the specific gravity will not be influenced by a change in the thickness of the water hull.

If the density of the adsorbed water is less than the density of the free water, then the specific gravity will decrease due to the decrease in W_3 which causes a larger denominator. In this situation the specific gravity will decrease with a decrease in salt content due to the influence of the adsorbed water.

The weight of the soluble salts will increase the specific gravity in each case, i.e., as the salt content increases the specific gravity will increase.

In the case of the sodium modification the specific gravity was observed to decrease with an increase of salts. In the light of prementioned facts in order for the specific gravity to change in the manner observed, the influence of the adsorbed water hull must overshadow the additional weight of the soluble salts. Also the density of the adsorbed water in this case must be greater than the free water. This latter fact is a point of great controversy (Sec. 6:2). The presence of a thick dense adsorbed water hull will decrease the denominator of the modified equation, thereby resulting in a greater value of the specific gravity. This increase must be large enough to overcome the increase in the numerator caused by the weight of the salts.

In the case of calcium modification the lack of variation in the specific gravity with salt content might be attributed to the fact that the influence of the adsorbed water hull balances the effect of the weight of the insoluble salts.

This discussion is presented in order to stimulate interest in the problem of specific gravity determinations. It is not intended to be a conclusive explanation but merely food for thought on the problem.

Physico-chemical phenomena appear to influence the observed determinations of specific gravity. The exact nature of the manner in which these phenomena affect the specific gravity is not readily apparent, however, the factors discussed undoubtedly contribute. The significance of the specific gravity is visualized when one realizes that the void ratio, which is of the utmost importance in shear strength evaluation, is based on the specific gravities determined.

7:4 Liquid Limits and Plastic Limits of Sodium and Calcium Modifications

Liquid limits were found to vary with salt content, but the variation in plastic limits seems to be within the limits of experimental error. The variation in liquid limits substantiates the fact that the change in shear strength with salt content is not a linear relationship with the sodium modification.

The degree of plasticity develops with the moisture content of a cohesive soil. The first addition of water to a fine grained soil develops the adsorbed water hull (Grim, 1962, p.220). Until this layer is fully developed there will be no free water available for lubricating action.

With the sodium-modification with zero-salts in the pore water, the adsorbed water hull is relatively large. Therefore, a great deal of water must be added before the layer is developed to the point where there is sufficient free water for the lubrication effect which is required to arrive at the liquid limit. As the salt content increases, the double layer becomes suppressed and liquid limit value decreases.

The same trend of thought serves to explain the variation in the liquid limits for the calcium modifications. The fluctuations in the calcium-clay were not as large as with the sodium-clay.

Figure 15 shows the relationships of the soil modifications on the plasticity chart. The sodium and calcium modifications exhibit an increase in dry strength and a decrease in permeability with a decrease in salt content. This figure also illustrates that physico-chemical influence on a soil can produce a relationship between liquid limit and plasticity index which is not parallel to the A-line.

CHAPTER VIII

CONCLUSIONS

8:1 Introduction

It appears that the following conclusions are justified in the light of the results obtained from the testing procedures and soils employed in this thesis.

8:2 Sodium Modification

1. The shear strength of the homionic sodium modification varies with the concentration of sodium salts in the pore water as indicated in Figures 2, 3, and 19. The major variation in the shear strength is in the effective angle of shearing resistance. The total angle of shearing resistance varies in the same manner but to a lesser degree. The cohesion values are small and have no consistent variation with the concentration of salts in the pore water. The variations in the angles of shearing resistance are not in linear proportion to the changes in salt content. In general, there is an increase in shear strength with an increase in salt content from zero to 8 me/100 gms, a.d.s. and then the values remain relatively constant up to the maximum salt content studied.

2. There is a marked decrease in the length of time to reach theoretical 100% consolidation with an increase of salts in the pore water. The decrease in t_{100} is approximately 170 times for an increase in salt content from 0.0 to 13.1 me/100 gms. a.d.s.

3. The plastic limit does not vary to any significant degree with a change in salt content.

4. The liquid limit decreases with an increase of salt content from zero to 5 me/100 gms. a.d.s. and then remains relatively constant.

5. Specific gravity is influenced by the concentration of salts in the pore water and the adsorbed cation complex.

8:3 Calcium Modifications

1. Within the range of salt concentrations studied, there is no consistent variation in the shear strength in terms of either angles of shearing resistance or cohesion.

2. There is a strong indication that t_{100} tends to increase with an increase in salt content.

3. The plastic limit remains essentially constant within the range of salt contents studied.

4. The liquid limit decreases with an increase in salt content.

5. The specific gravity does not vary significantly with the salt content, but the calcium modification has a lower specific gravity than the sodium modification.

8:4 General

In general it may be stated that the results of this thesis follow the trends and substantiate the facts revealed in previous research of this nature performed at the University of Alberta. The procedures employed to produce homionic soil were refined slightly and found to be successful in the light of this research. The data substantiates previous research on Atterberg limits. (Thomson, 1960, Hamilton, 1961). The

with respect to shear strength and consolidation substantiate the work of Thomson (1963).

CHAPTER IX

RECOMMENDATIONS

9:1 General

From the research performed it seems justified to present some recommendations which may prove useful in future research of this nature.

9:2 With Respect to the Testing Programme

1. Immediately after a soil has been modified, it should be checked by flame photometer and titration procedures to determine the concentration of salts in the pore water and the adsorbed cation complex. This procedure would obviate future difficulties in the control of the homionic state and the pore water concentration.

2. Remoulded triaxial samples should be wrapped in wax paper before wrapping in aluminum foil. This procedure would prevent cation exchange with the aluminum ions of the foil.

9:3 With Respect to Future Research

1. Research on the same basic problem should be performed using homionic modifications of other cations found in nature, eg. hydrogen, magnesium, and potassium. From the viewpoint of research lithium and barium could also be investigated.

2. Investigations of the effect of one cation upon another could also be performed, e.g. various percentages of Ca^{++} and Na^{+} adsorbed, various percentages of Ca^{++} and Mg^{++} adsorbed.

3. The effect of anions on physical properties should also be investigated; e.g. Cl^- , PO_4^{+++} , SO_4^{++} . These anions may be found in the adsorbed state or present in the pore water. Lambe has stated that adsorbed anions can influence the shear strength of a soil.

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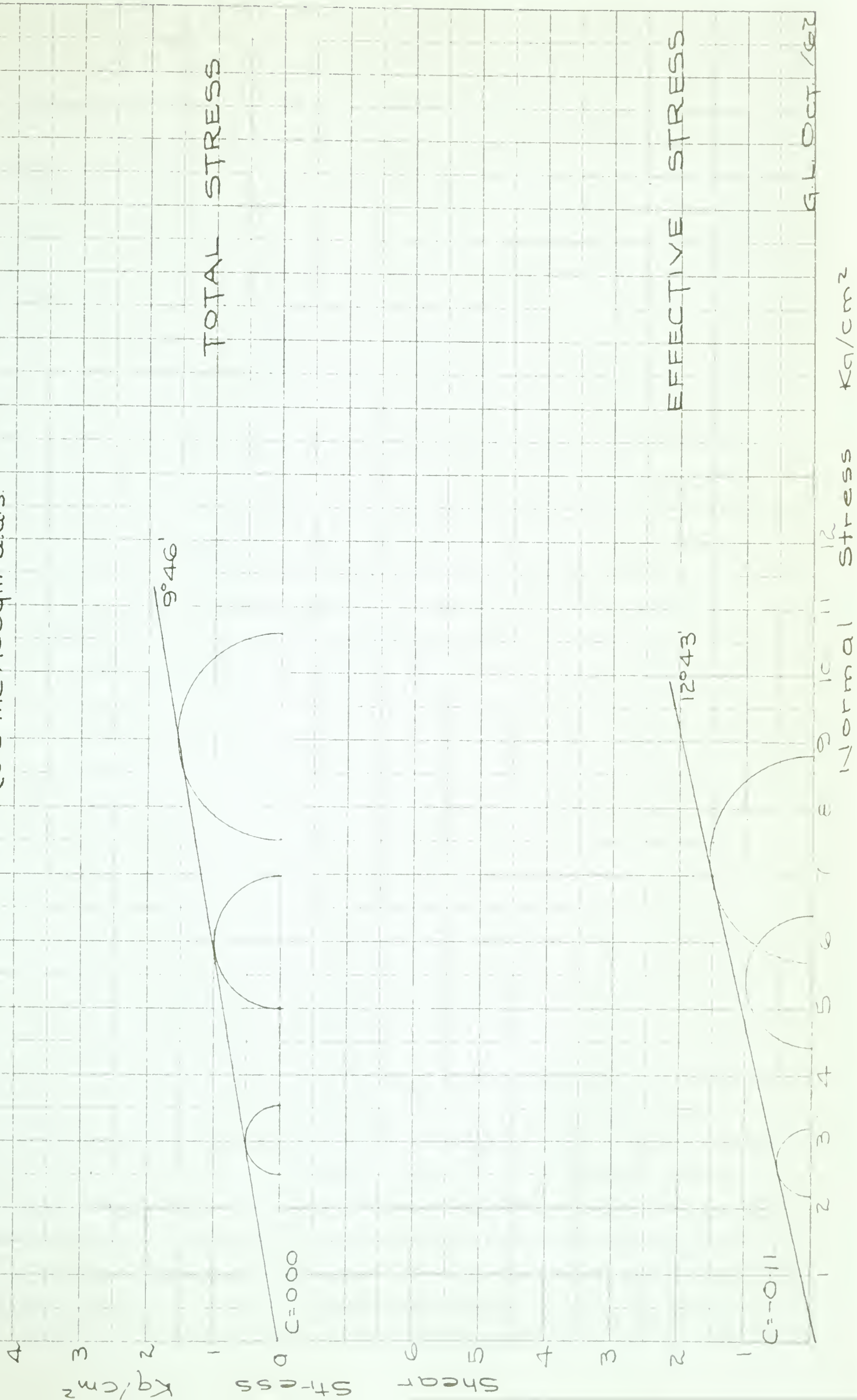
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APPENDIX A

MOHR ENVELOPES

MOHR ENVELOPE REMOLDED EDMONTON CLAY SODIUM MODIFICATION

{ 0.0N SODIUM SALTS IN PORE WATER
 { 0.0 me / 100qm a.d.s.



MOHR ENVELOPE
REMOLDED EDMONTON CLAY
SODIUM MODIFICATION
{ 0.1N SODIUM SALTS IN PORE WATER
2.3 me/100gm a.d.s.

TOTAL STRESS

EFFECTIVE STRESS

G.L. DIST / 62

Kg/cm²

Normal Stress

1 2 3 4 5 6 7 8 9 10 11 12

6

5

4

3

2

1

6

5

4

3

2

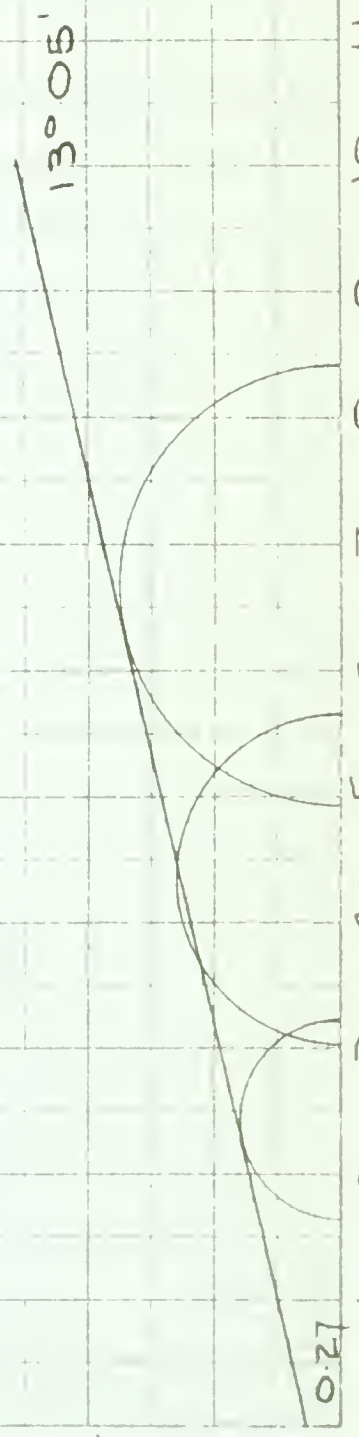
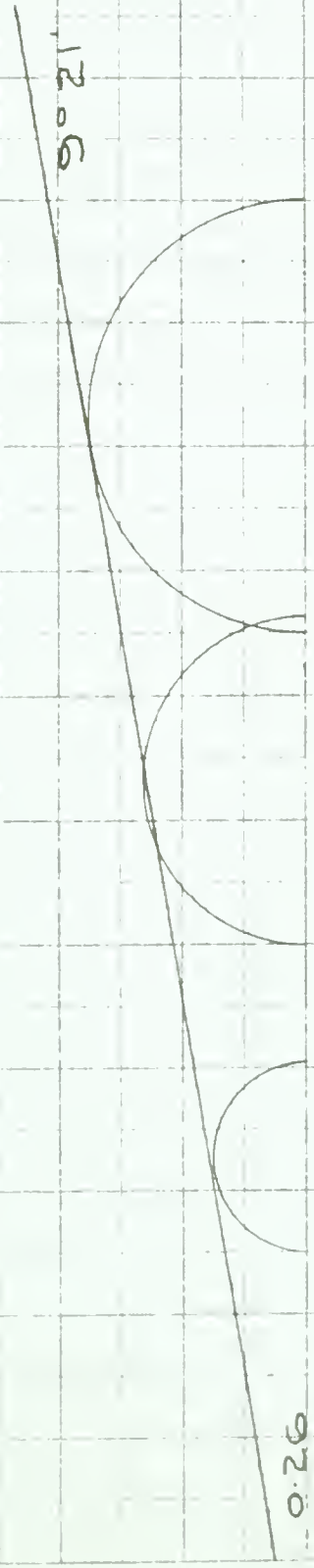
1

Shear Stress Kg/cm²

Shear Stress Kg/cm²

Shear Stress Kg/cm²

Shear Stress Kg/cm²



MOHR ENVELOPE
 REMOLDED EDMONTON CLAY
 SODIUM MODIFICATION
 { 0.2N SODIUM SALTS IN PORE WATER
 { 4.7 me / 100 gm ads

TOTAL STRESS

EFFECTIVE STRESS

C=0.22

C=0.31

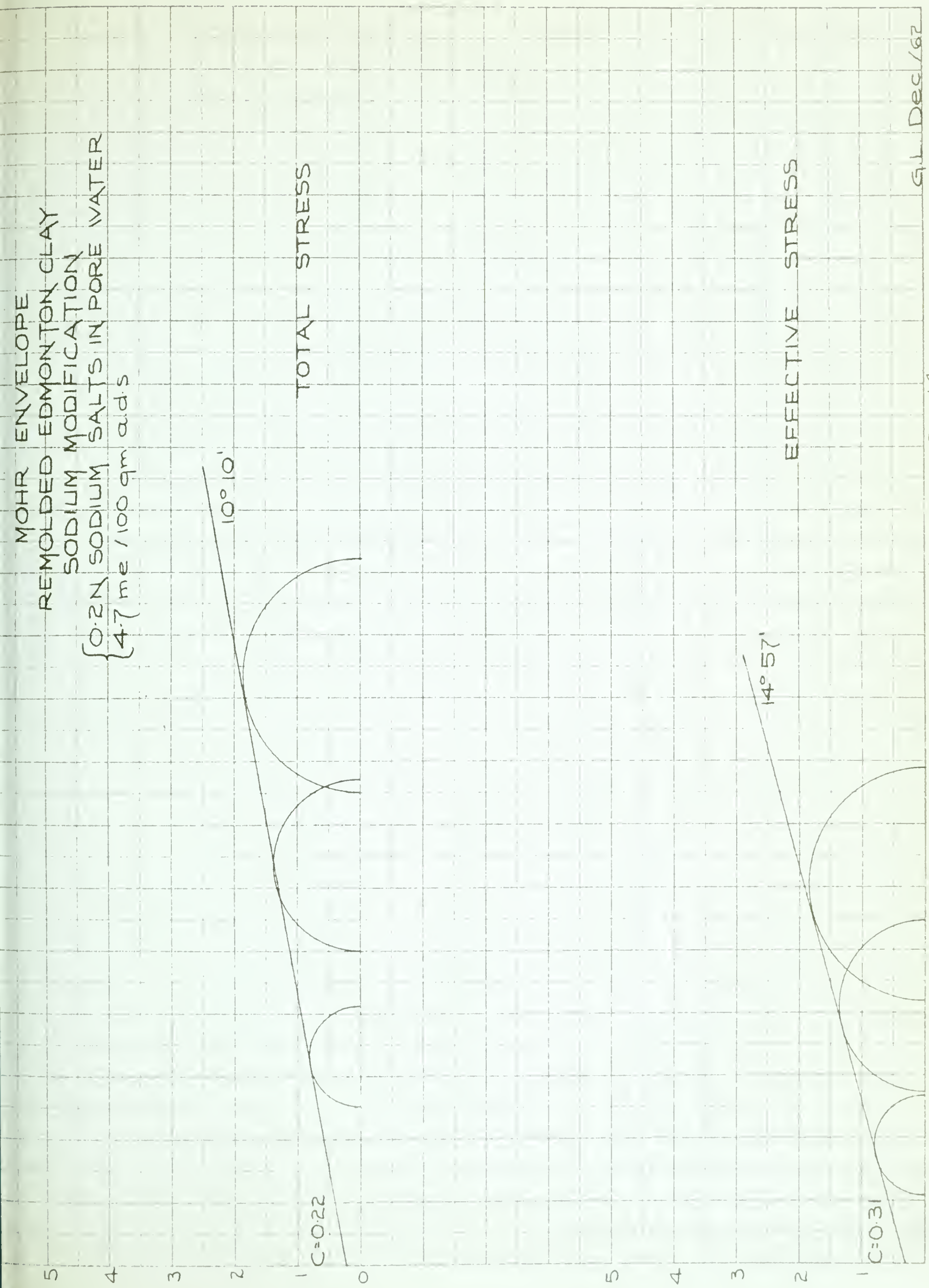
10° 10'

14° 57'

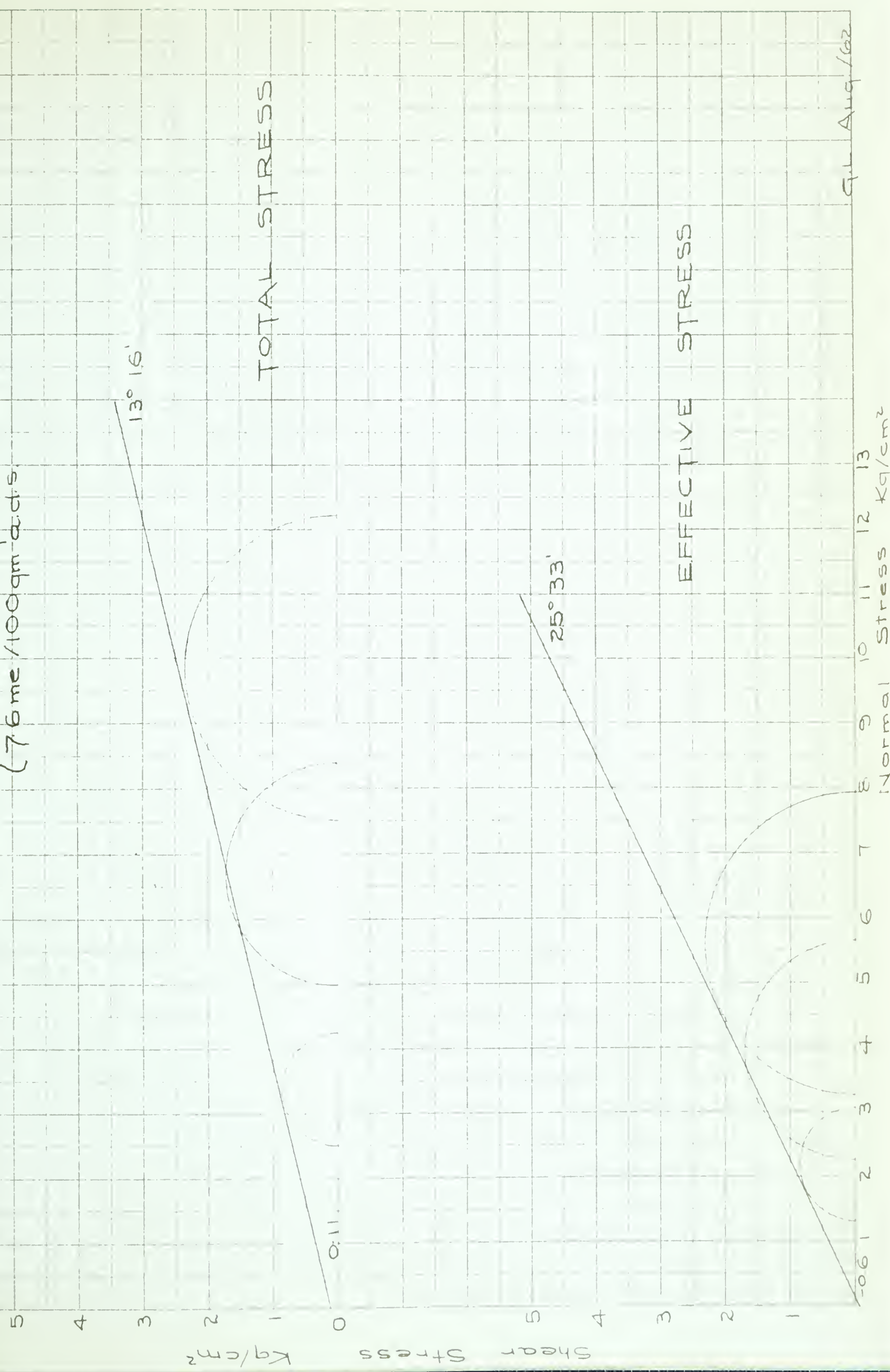
GL Dec 62

Normal Stress Kg/cm²

Shear Stress Kg/cm²



MOHR ENVELOPE
REMOLDED EDMONTON CLAY
SODIUM MODIFICATION
{ 0.3N SODIUM SALTS IN PORE WATER
{ 7.6 me / 100 gm c.c.s



g.l. Aug. 1962

MOHR ENVELOPE
 REMOLDED EDMONTON CLAY
 SODIUM MODIFICATION
 { 0.5N SODIUM SALTS IN PORE WATER
 { 13.1 me / 100 qm a.c.s

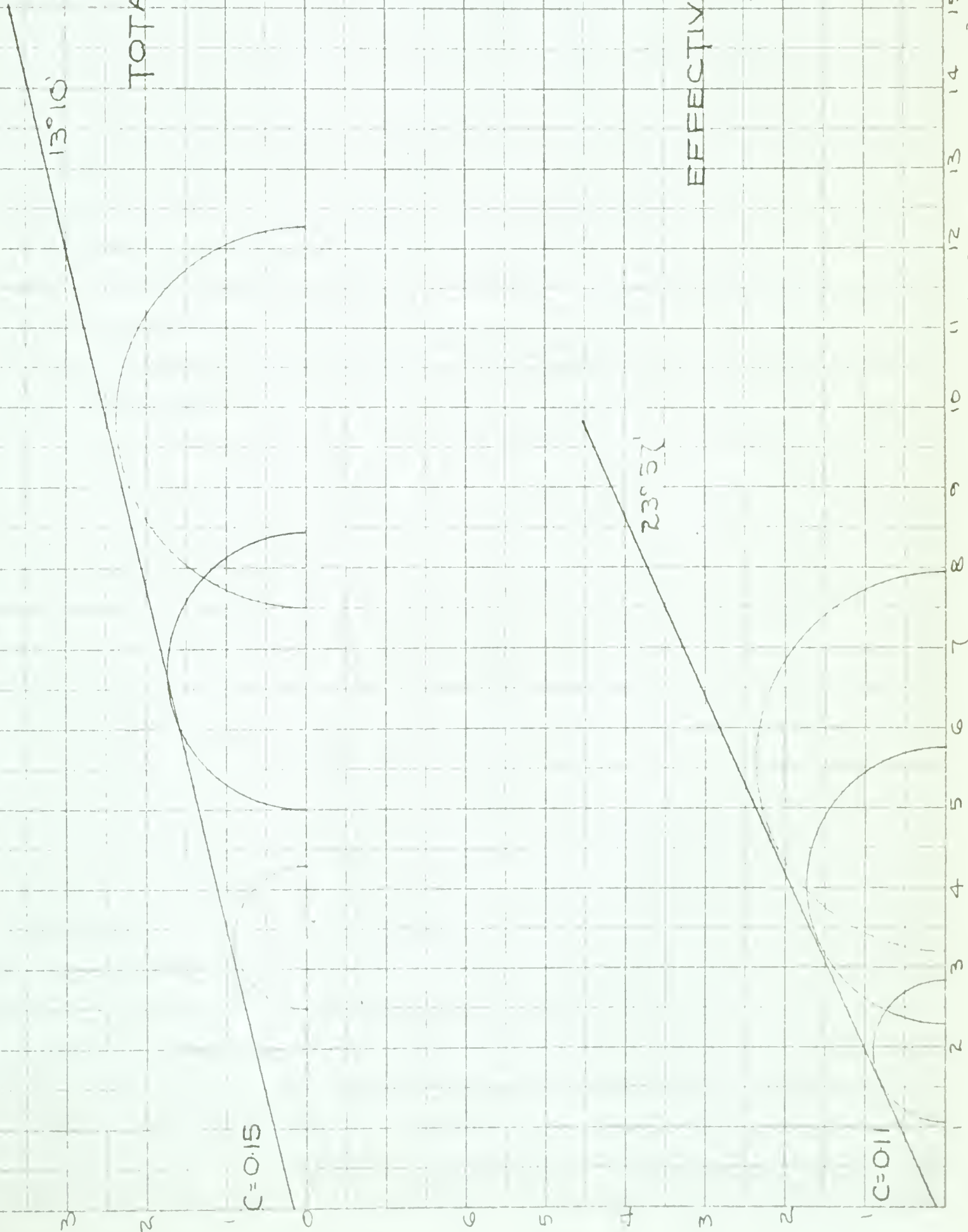
TOTAL STRESS

EFFECTIVE STRESS

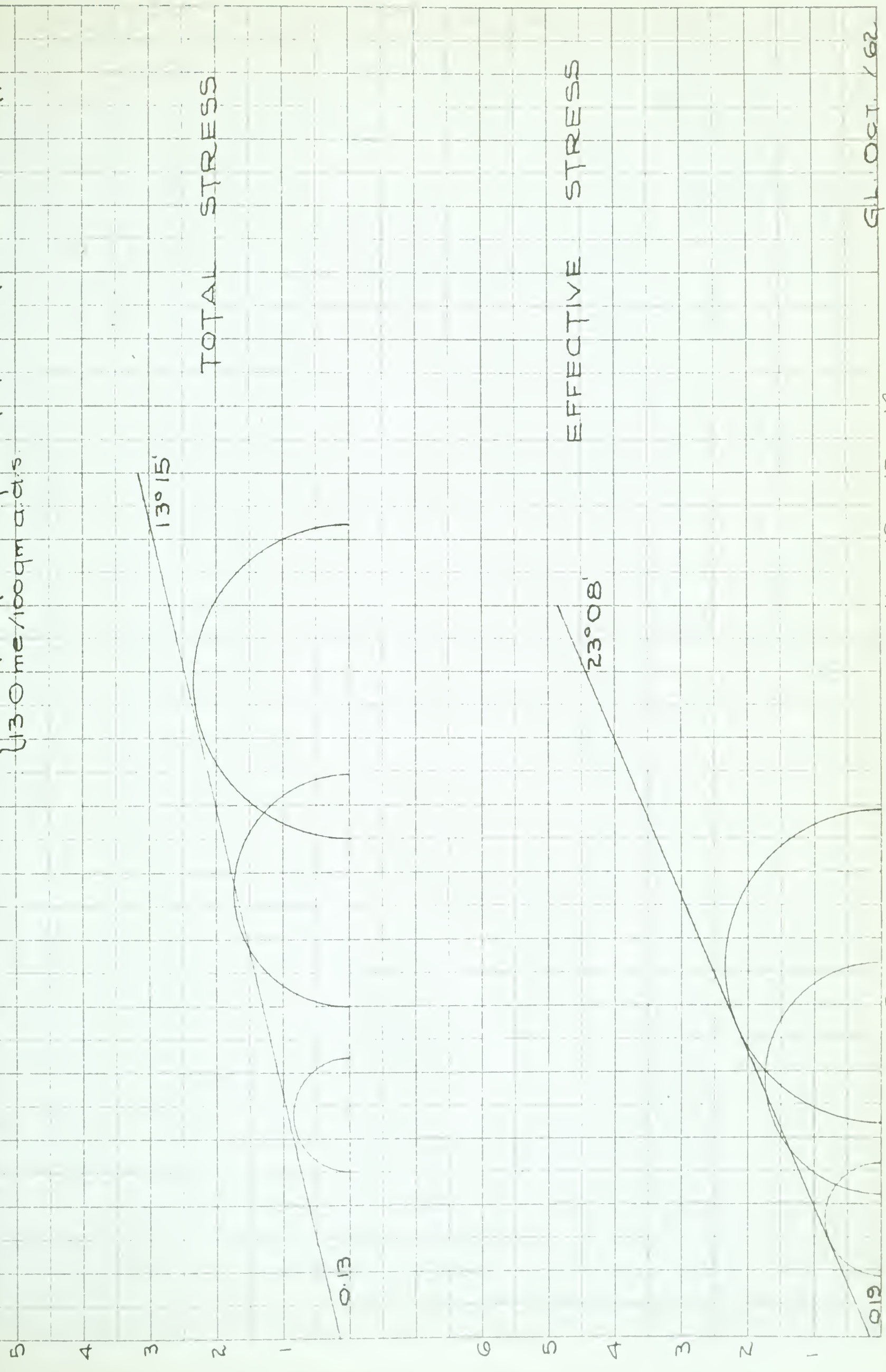
Shear Stress Kg/cm²

Normal Stress Kg/cm²

Q.L. DEC 1962



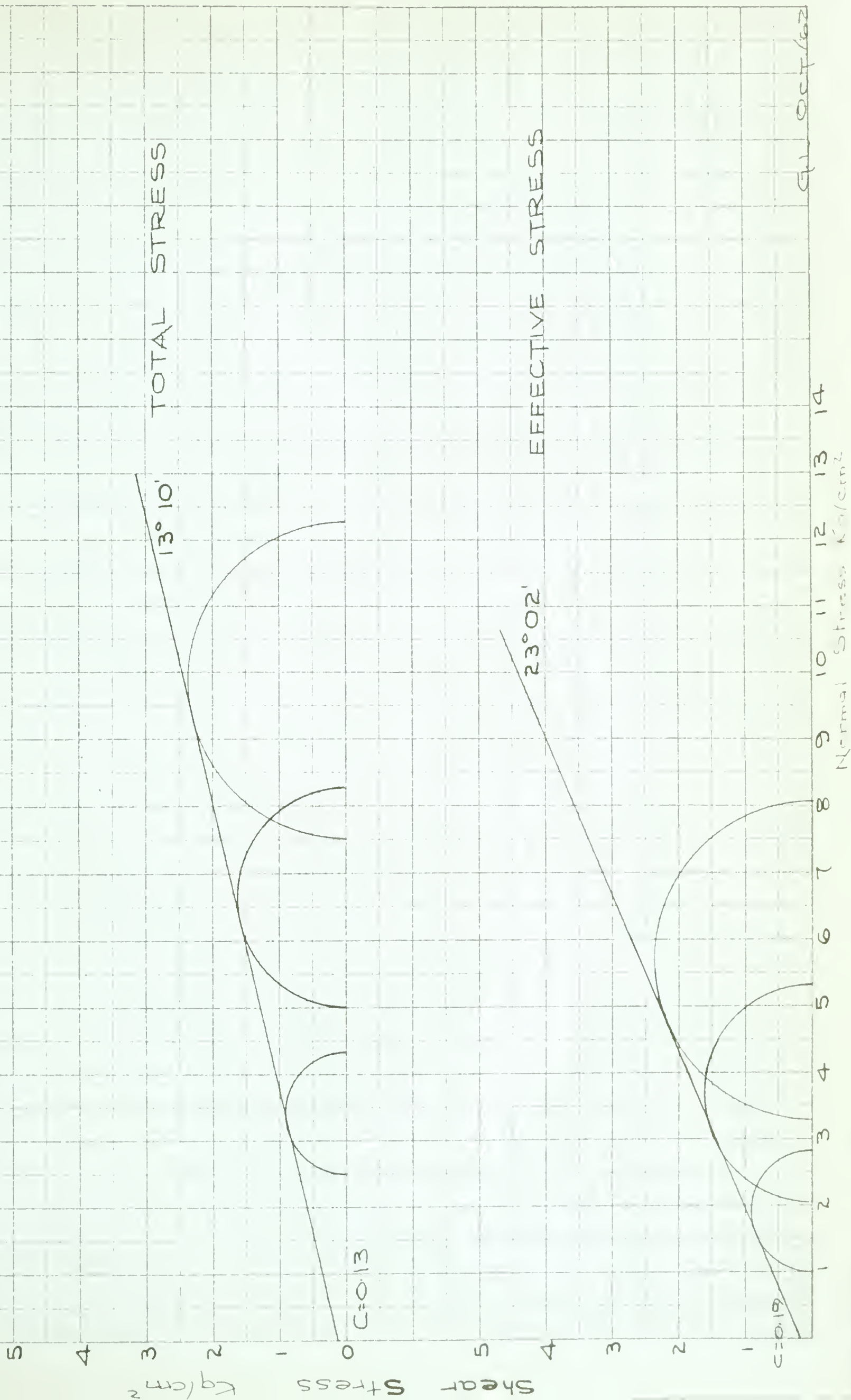
MOHR ENVELOPE
 REMOLDED EDMONTON CLAY
 CALCIUM MODIFICATION
 { 0.0N CALCIUM SALTS IN PORE WATER
 { 13.0 me/100gm a.d.s.



Normal Stress Kg/cm^2

GL: OCT. 1962

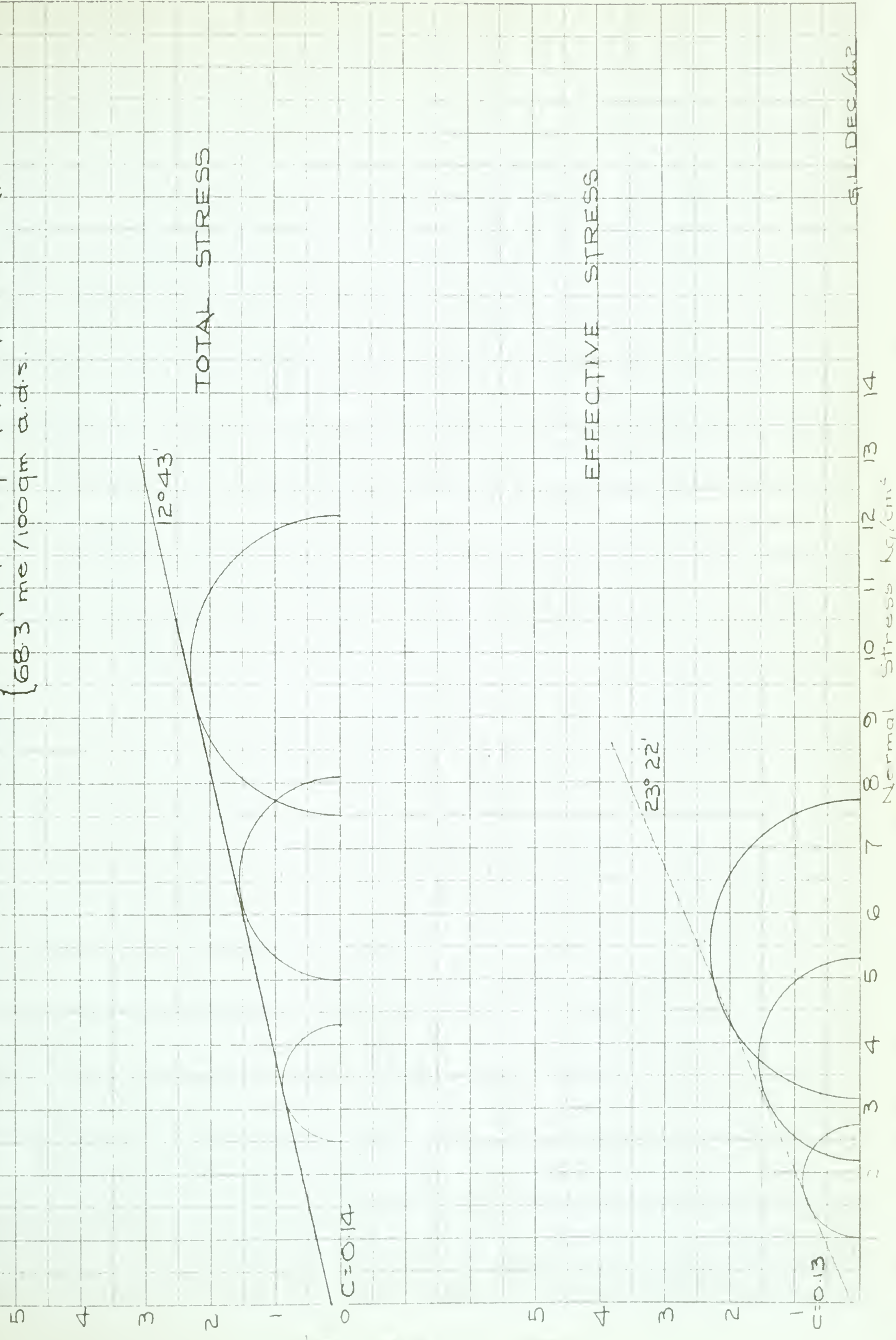
MOHR ENVELOPE
 REMOLDED EDMONTON CLAY
 CALCIUM MODIFICATION
 { 0.3N CALCIUM SALTS IN PORE WATER
 { 18.9 me / 100qm a.s.



MOHR ENVELOPE
 REMOLDED EDMONTON CLAY
 CALCIUM MODIFICATION
 { 0.8N CALCIUM SALTS IN PORE WATER
 { 68.3 me/100qm a.d.s.

TOTAL STRESS

EFFECTIVE STRESS



G.L. DEC 1962

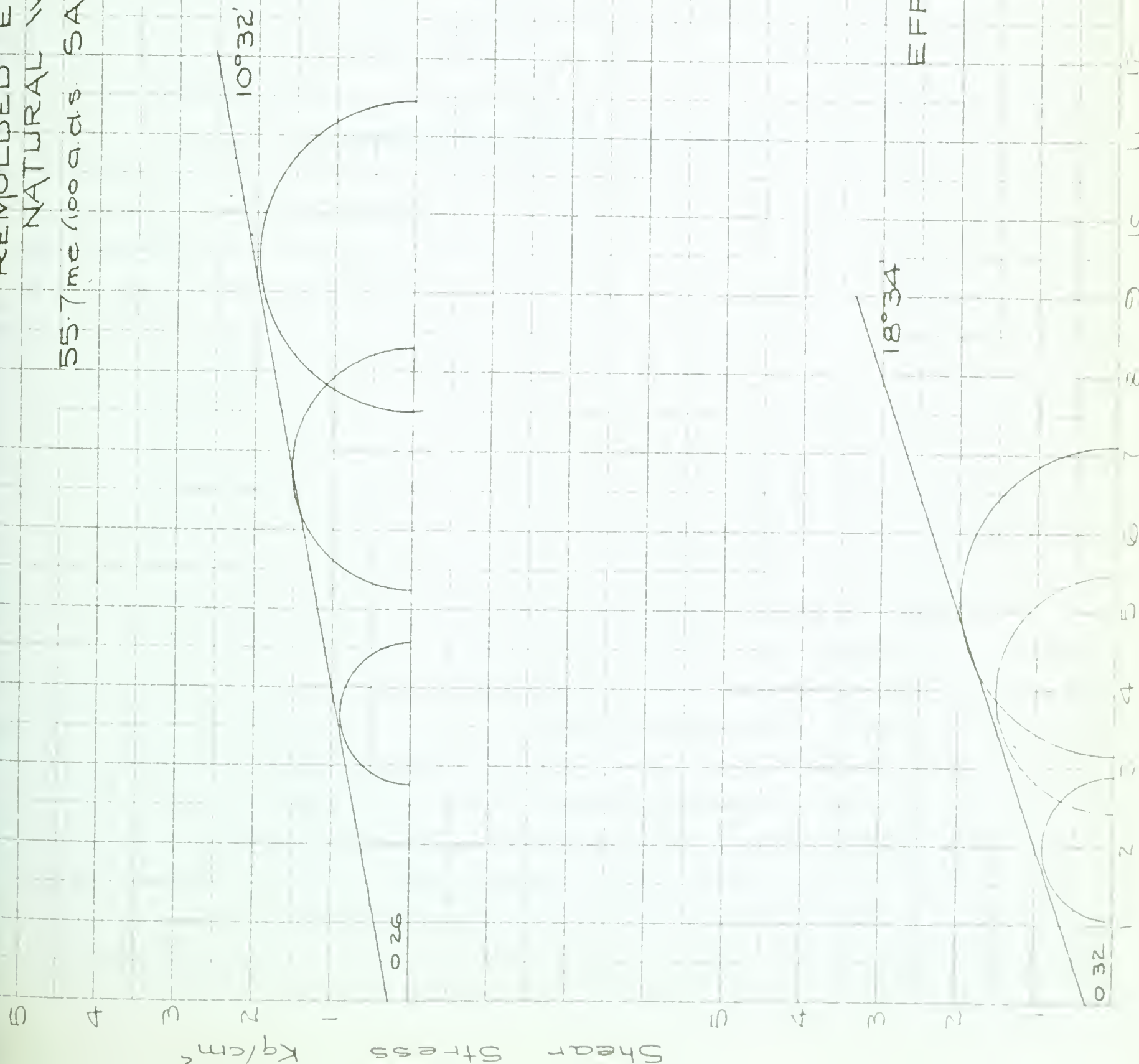
MOHR ENVELOPE
 REMOLDED EDMONTON CLAY
 NATURAL WASHED CLAY
 55.7 me 100 a.d.s. SALTS IN THE PORE WATER

TOTAL STRESS

EFFECTIVE STRESS

G.L. JUNE 1962

Normal Stress kg/cm²



APPENDIX B

MOUNTING THE TRIAXIAL TEST SPECIMEN

APPENDIX B

MOUNTING THE TRIAXIAL TEST SPECIMEN

The following is the sequence of steps used in mounting the tri-axial test specimens:

1. The sealing wax and aluminum foil were removed from the sample.
2. The specimen was trimmed to 35.7 mm in diameter by 80 mm long with the aid of the Geonor trimming accessories.
3. Filter paper, porous plates, and wool wicks were all soaked in the solution employed for the formation of the sample being mounted.
4. Five wool wicks were inserted lengthwise into the sample to within one centimeter of the top. A lucite template was used to provide symmetrical spacing of the wicks; one wick was placed at the center; and the other four were equally spaced around it at a distance half-way between the center and the outer periphery.
5. Specimen dimensions were taken with calipers to the nearest 0.1 mm. The length was taken as the average of two measurements and the diameters at the top, center, and bottom were each taken as the average of two measurements, each one at 90 degrees to the other.

The initial cross-sectional area was computed as:

$$A_O = 1/4 (A_T + 2A_C + A_B)$$

where: A_O denotes initial cross-sectional area

A_T denotes average area of the top

A_C denotes average area of center

A_B denotes average area of bottom

6. The sample was weighed to the nearest milligram.
7. The pore pressure lines on the base of the triaxial cell were flushed with distilled water from an attached 25 millilitre stopcock burette. The lines were left full by closing off one line with a tapered steel pin and leaving the other line attached to the burette.
8. A saturated porous ceramic disc and filter paper were placed in contact with the bottom of the specimen and a saturated slotted filter paper was wrapped around the specimen so that it was in contact with the porous disc. The sample was set carefully on the pedestal which had been encased in a piece of rubber tubing in such a manner that the tubing covered the porous disc.
9. The top loading head which was also encased in a piece of rubber tubing was centered on the specimen. Two rubber membranes were placed around the specimens with the aid of a membrane stretcher. A thin layer of silicone grease (Dow Corning high vacuum grease) was placed between the two membranes and on the outside of the outer membrane.
10. The membranes were secured to the pedestal and the loading cap with two rubber "O" rings at each location.
11. The triaxial cell was secured in position and the chamber filled with water; leaving sufficient room at the top for an oil seal. The thickness of the oil seal depended on the estimated length of time required for consolidation, e.g. about a one inch seal was required for a sample requiring one to two months for consolidation. The soil (Esso Teresso No. 65) acted as the seal to maintain pressure in the cell.

12. The loading piston was placed and clamped.
13. The free air in the cell was removed by forcing it out the top valve with pressure supplied by the screw control. The pore pressure lines were flushed once again with distilled water to remove any air present.
14. With the burette shut off, the cell pressure was built up to the desired confining pressure and the constant pressure cell was brought into the system.
15. The burette was opened and a stop clock started. In this manner typical time curves of burette reading vs. logarithm of time were obtained.
16. Consolidation was carried out to at least theoretical 100 percent consolidation.
17. After consolidation the burette stopcock was closed and the cell was mounted in the loading press.

APPENDIX C

REPRESENTATIVE TRIAXIAL TEST DATA SHEETS

AND

SAMPLE CALCULATIONS

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. _____

Depth _____

Sample 2-3Engineer G.L.

Technician _____

Date of Test 12 OCT. /62Test Lateral Pressure σ_3 5.0 Kg/cm²Back Pressure 2.0 Kg/cm²Remarks N.A. (0.1 N) (2.3 me/100gm
L = 80.67 (a.c.s.)Area Correction Factor 1.215 (19.97)

at Failure

 $\sigma_3 = 2.62$ $\sigma_1 = 7.62$ 1.955.673057.5

me in	Strain Dial Div.	A _c cm ²	No. of Stress Dial Div.	Proving Ring Const δ , kg/Div	$\sigma_1 - \sigma_3$ $= \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm ² P _p	Effective Stress		Stress Ratio $\frac{\sigma_1}{\sigma_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{\bar{P}_p}$ $\frac{A}{\sigma_1 - \sigma_3}$
0	0	10.22	0								
2	3	10.22	20	0527	0.125	0					
3	5	10.22	40	0510	0.243	0.02					
5	6	10.23	60	0493	0.351	0.06					
		10.23	80	0484	0.460						
7	8	10.23	94	0488	0.54	0.22	5.32	4.78	1.112	1	407
17	16	10.24	200	0463	1.10	0.50	5.60	4.50	1.244	2	455
24	24	10.25	260	04596	1.40	0.59	5.81	4.41	1.317	3	421
30	32	10.26	304	04500	1.62	0.74	5.88	4.26	1.380	4	457
35	40	10.27	337	04472	1.78	0.80	5.98	4.20	1.424	5	449
38	48	10.28	362	04449	1.90	0.88	6.02	4.12	1.461	6	463
42	56	10.29	385	04431	2.01	0.93	6.08	4.07	1.494	7	463
46	64	10.30	401	04420	2.09	0.97	6.12	4.03	1.519	8	464
49	72	10.31	415	04410	2.16	0.99	6.17	4.01	1.539	9	458
53	81	10.32	430	04400	2.23	1.06	6.17	3.94	1.566	10	475
57	89	10.33	441	04394	2.28	1.10	6.18	3.90	1.585	11	482
60	97	10.34	450	04390	2.32	1.12	6.20	3.88	1.598	12	482
64	105	10.35	457	04383	2.35	1.15	6.20	3.85	1.610	13	489
68	113	10.36	465	04380	2.39	1.17	6.22	3.83	1.624	14	490
72	121	10.37	471	04378	2.42	1.18	6.24	3.82	1.634	15	488
76	129	10.38	477	04372	2.44	1.20	6.24	3.80	1.642	16	492
80	137	10.39	482	04370	2.46	1.26	6.20	3.74	1.658	17	512
84	145	10.40	487	04366	2.48	1.27	6.21	3.73	1.665	18	512
88	153	10.41	492	04364	2.51	1.28	6.23	3.72	1.674	19	510
92	161	10.42	496	04361	2.51	1.30	6.21	3.70	1.678	20	518
96	177	10.44	503	04359	2.55	1.34	6.21	3.66	1.697	22	525
100	194	10.47	510	04354	2.68	1.38	6.20	3.62	1.713	24	535
104	210	10.49	515	04350	2.59	1.43	6.16	3.57	1.725	26	552
108	226	10.52	520	04349	2.61	1.44	6.17	3.56	1.739	28	552

Department of Civil Engineering

Soil Mechanics Laboratory

AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. _____

Depth _____ Sample 2-3Area Correction Factor 1.215

Time min	Strain Dial Div.	A_c cm ²	No. of Stress Dial Div.	Proving Ring Const δ , kg/Div	$\sigma_1 - \sigma_3$ $= \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm ² P_p	Effective Stress		Stress Ratio $\frac{\sigma_1}{\sigma_3}$	Axial Comp. Strain %	\bar{A} $\frac{P_p}{\sigma_1 - \sigma_3}$
							$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor			
4	242	10.54	523	0.4349	2.62	1.48	6.14	3.52	1.744	3.0	565
20	262	10.57	528	0.4345	2.64	1.52	6.12	3.48	1.759	3.25	576
	282	10.59								3.50	
34	303	10.62	536	0.4340	2.66	1.60	6.06	3.40	1.782	3.75	602
41	323	10.65	539	0.4339	2.67	1.64	6.03	3.36	1.795	4.0	614
59	363	10.70	543	0.4338	2.67	1.65	6.02	3.35	1.797	4.5	618
67	403	10.76	546	0.4337	2.67	1.75	5.92	3.25	1.822	5.0	655
80	444	10.81	548	0.4336	2.67	1.78	5.89	3.22	1.829	5.5	667
95	484	10.87	550	0.4335	2.66	1.85	5.81	3.15	1.844	6.0	695
107	524	10.93	550	0.4335	2.65	1.87	5.78	3.13	1.847	6.5	706
120	565	10.99	550	0.4335	2.64	1.92	5.72	3.08	1.857	7.0	727
135	605	11.05	549	0.4335	2.62	1.95	5.67	3.05	1.859	7.5	744
146	645	11.11	544	0.4338	2.58	1.98	5.60	3.02	1.854	8.0	767
160	686	11.17	533	0.4341	2.52	2.00	5.52	3.00	1.840	8.5	794
172	726	11.23	507	0.4356	2.39	2.00	5.39	3.00	1.797	9.0	837
	766	11.30								9.5	
	807	11.36								10.0	
	847	11.42								10.5	
	887	11.48								11.0	
	928	11.54								11.5	
	968	11.61								12.0	
	1008	11.67								12.5	
	1049	11.74								13.0	
	1089	11.81								13.5	
	1129	11.88								14.0	
	1170	11.95								14.5	
	1210	12.02								15.0	
	1250	12.10								15.5	
	1291	12.17								16.0	

Moisture Content - Initial _____

Final 44.42%

Sketch of Failure

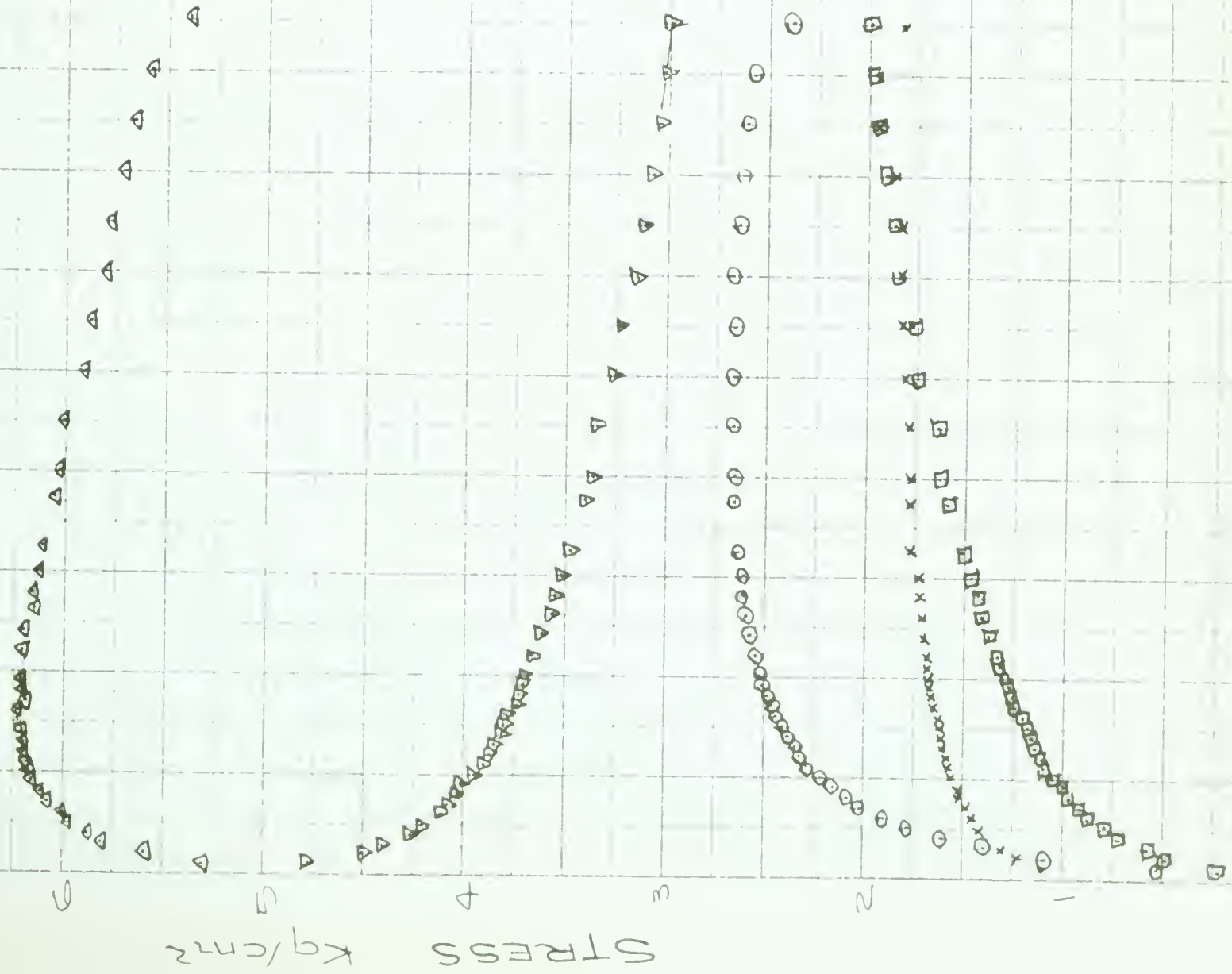
Degree of Saturation - Initial 96.72%Final 96.94% $L = 70.44$ $T = 33.30$ $C = 33.72$ Liquid Ratio - Initial 1.97Final 1.29 $B = 34.50$ Pressure Reaction 90 / 2 m.

TRIAXIAL COMPRESSION TEST

Q_c TEST

Remolded Edmonton Clay {Nat (0.1N)
Sample N^o 2-3 (2.3mc/100gm a.d.s)

$$\sigma_3 = 5.0 \text{ Kg/cm}^2$$



Δ Major Principal Effective Stress
 ▽ " " "
 ○ Deviator Stress
 □ Pore Pressure
 x Principal Effective Stress Ratio

% STRAIN

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DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST
PORE PRESSURE REACTION TEST

Sample No. 2-3

Sample Desc. NA⁺ (OIN)

Kg/sq. cm on cell 1

SC = 23 me/100gm
ads

"LOAD"

t min	Pp kg/cm ²
0	
0.1	50
0.25	90
0.50	90
1.0	90
2	90
3	
4	
5	

"UNLOAD"

t min	Pp Kg/cm ²
0	
0.1	
0.25	12
0.50	10
1.0	08
2	06
3	
4	
5	

Final Moisture Content

Wet Wt. plus tare 98.4690

Dry Wt. plus tare 88.2440

Wt. of water 10.2250

Wt. and No. of tar 65.2272

Wt. of dry soil 23.0168

Final M.C. 44.42

Final Volume

Wt. Hg + tare 627.97 + 670.38 = 1298.35

Tare 436.23

Wt. Hg 862.12

Temp 25.0°C (13.534)

Vol. Hg 6370

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DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

Sample No. 2-3
Sample Desc. NAT (CLN)

SC - 23 me/100
grn ads

BEGINNING OF TEST

Original Vol of Specimen 82.44
Wt. of Soil Solids in Specimen 18.36
Vol. of Soil Solids 27.79
Vol of Voids 54.65
Original Void Ratio, e_0 1.97
Original Porosity, n , 0.66
Wt. of Water 52.86
Original Degree of Saturation, S , 96.72%
Original Wet Wt., lb/cu. ft. 99.37
Original Dry Wt., lb/cu. ft. 59.24

END OF TEST

Final Vol of Specimen (by Hg immersion) 63.75
Vol. of Soil Solids 27.79
Vol. of Voids 35.91
Final Void Ratio, e_f 1.29
Final Porosity, n , 0.5
Wt. of Water 34.81
Final Degree of Saturation 96.91%
Final Wet Wt. lb/cu.ft. 110.91
Final Dry Wt. lb/cu.ft. 76.80
Wet wt. of specimen at beginning of test 131.215 gm.
Wet wt. of specimen at end of test 113.1704 gm.
Weight loss 18.0446 gm.
Vol. change from burette rdg. discrepancy 1.93 cc.

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAxIAL COMPRESSION TEST ON COHESIVE SOIL

2.3 mc/100 g m.a.d.s. Project THESIS

Sample Description NA⁺ (0.1 N)

Hole No. _____

Remolded Ed. Clay

Depth _____ sample 2-3

W5 + T 133.840

Engineer G.L.

2.625

Technician D.W.

Initial Weight 131.215 gms

Date of Set-up 7 SEPT. 1962

Test Lateral Pressure 5.0 kg/cm²

Length, mm 1. 80.74 2. 80.60

Aver: 80.67 Area Top A_T 10.13 sq.mm

Diam. mm Top 1. 36.00 2. 35.84

Aver: 35.92 Area Centre A_C 10.23 sq.mm

Centre 1. 36.04 2. 36.16

Aver: 36.10 Area Bottom A_B 10.31 sq.mm

Bottom 1. 36.40 2. 36.16

Aver: 36.28 Average X-Sect Area

Original Volume 82.44 cc

$$= \frac{1}{4} (A_T + 2A_C + A_B)$$

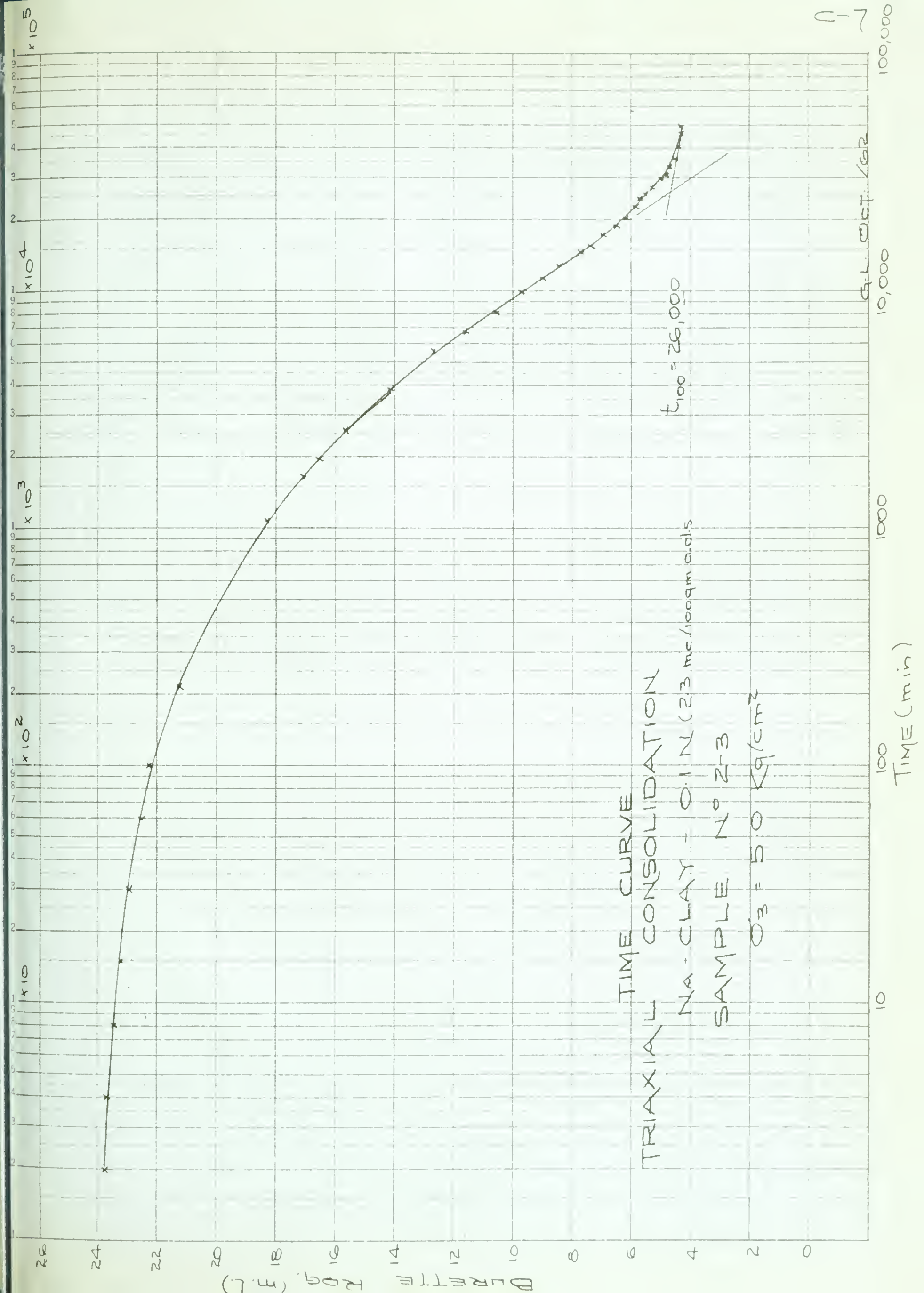
$$= \underline{10.22}$$

CONSOLIDATION DATA

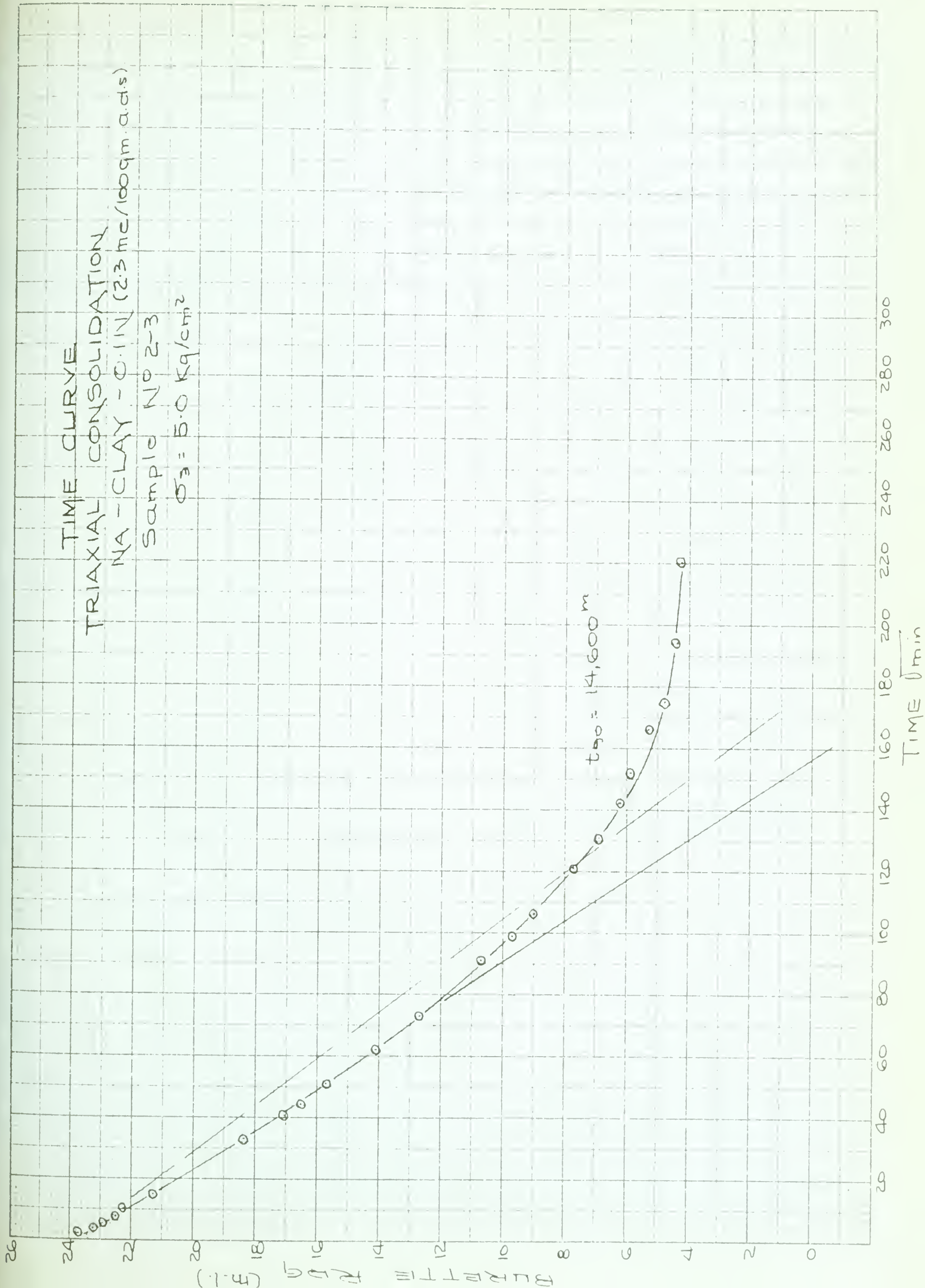
CONSOLIDATION DATA - cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
8:15:40	0	24.20	
	1	23.80	
	2	23.75	
	4	23.65	
	8	23.45	
	15	23.20	
	30	22.95	
	60	22.50	
	100	22.28	
9:15	215	21.30	
9:40	1080	18.38	
10:30	1670	17.11	
11:00	1970	16.57	
11:30	2600	15.66	
12:00	3890	14.10	
12:30	5300	12.70	
13:00	6710	11.60	
13:30	8150	10.65	
14:00	9800	9.70	
14:20	11,320	9.00	

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
16-14:35	12,825	8.40	
17-21:30	14,690	7.70	
18-14:00	15,680	7.40	
19-17:00	17,260	6.90	
20-16:30	18,670	6.51	
21-16:15	21,280	6.20	
22-11:30	20,095	6.20	
23-15:20	21,250	6.20	
24-10:30	22,920	5.90	
25-10:30	24,070	5.70	
26-17:00	25,510	5.50	
27-17:00	27,340	5.30	
28-11:00	29,860	4.90	
29-20:00	30,840	4.80	
Oct 1 9:00	33,060	4.72	
3 21:00	36,660	4.45	
4 12:00	37,560	4.40	
6 11:00	40,380	4.40	
9 22:00	45,340	4.30	
11 19:00	48,040	4.23	



TIME CURVE
TRIAXIAL CONSOLIDATION
NA - CLAY - 0.11N (2.3mc/100gm a.d.s)
Sample No 2-3
 $\sigma_3 = 5.0 \text{ Kg/cm}^2$



DEPT OF CIVIL ENGINEERING

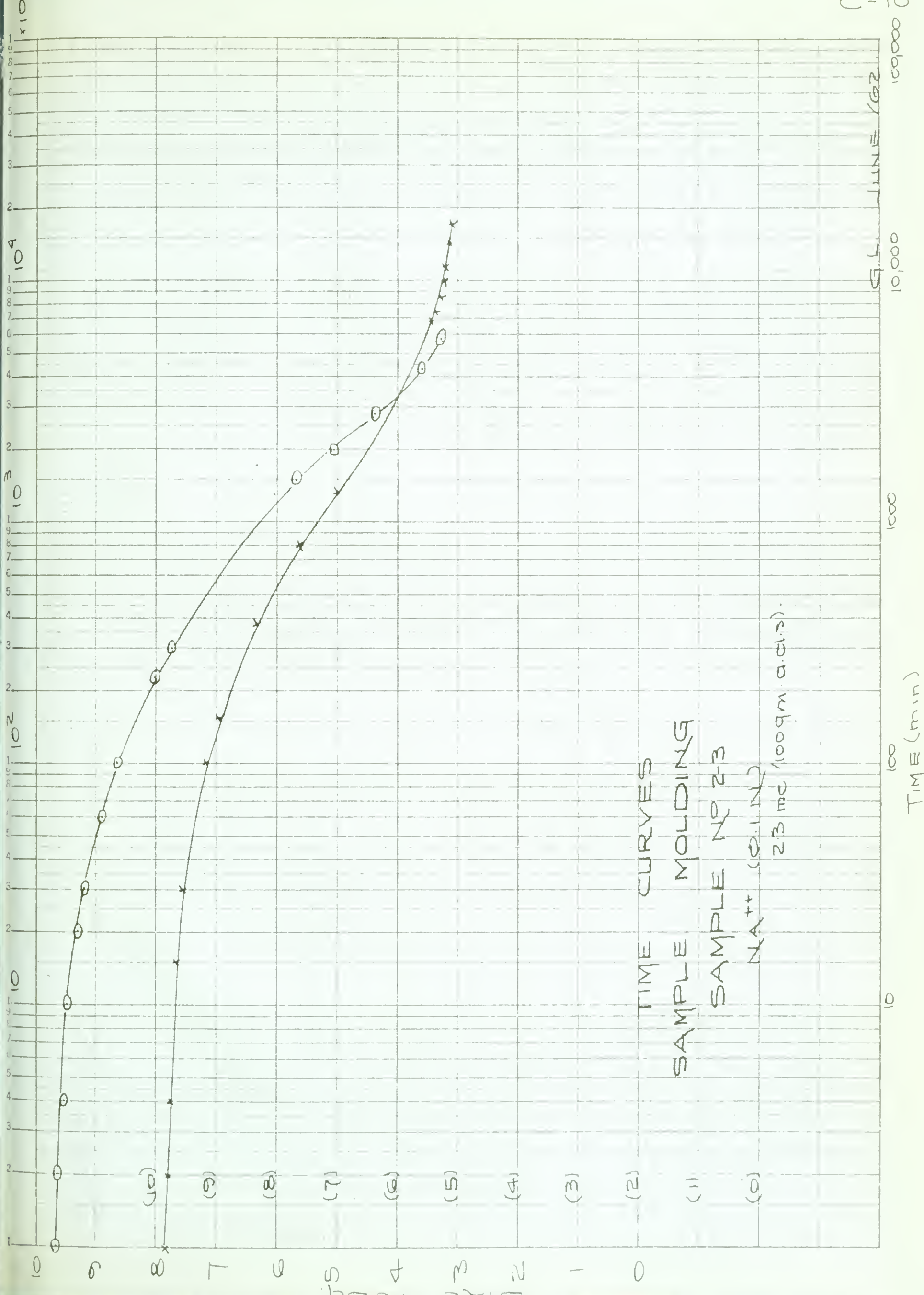
SOIL MECHANICS LABORATORY

CONSOLIDATION DATA

Engineer: G.L.
 Technician: D.W.
 Sample No: 2-3
 Date: 29 JUNE
 Remarks: NAT (0.1N) (2.3 mc/100gm)
 No. of tests: 12
 a.c.s.

C-9

Time	Time	At min	Dist mm	Notes	Time	At min	Dist mm
29 JUNE	10:50	0	1000			100	917
		1	966		13:15	155	895
		2	962		17:00	380	834
		4	956		24:00	800	762
		10	945	JULY 4	8:50	1330	701
		20	931	5	16:45	2245	657
		30	920	6	16:00	4640	562
		60	891	7	12:00	5840	547
		100	864	8	14:00	7400	536
		225	800	9	8:45	8525	530
		300	775	10	8:45	9965	525
30 "	11:40	1490	566	11	8:45	11,405	521
	20:30	2020	507	13	13:45	14,585	514
1 JULY	9:30	2800	438	15	15:10	17,550	512
2 "	10:30	4300	361				
3	08:30	5620	327				
"	10:40	0	1000				
		1	983				
		2	980				
		4	977				
		15	967				
		30	955				



0 1 10 100 1000 10000 100000

0 1 2 3 4 5 6 7 8 9 10

100

100

10

TIME (min)

0 1 2 3 4 5 6 7 8 9 10

100

1000

10000

100000

B29806